DESIGN OF STEEL STRUCTURAL ELEMENTS
(18CV61)

MODULE 01 – CHAPTER 01
INTRODUCTION

MODULE - 1

CHAPTER - 1 INTRODUCTION

A structure is the main part of a building which resist the forces due to usage & self weight. These structures can be built with bricks, reinforced cement concrete or steel or combination of these materials.

Steel as a construction material is commonly used in structures like power house, steel mill buildings, factories, workshops, warehouses, multi-story buildings, exhibition pavilians, domes, radio & TV towers, transmi-ssion towers, tanks etc. Hence steel is an important civil engineering materials.

Steel is an alloy of iron & carbon manuf-actured under control environment in factories.

The steel used for structural purposes is called structural steel. The content of carbon vary from 0.01% is called low carbon steel or mild steel. The steel containing upto 0.25% of carbon called high carbon or high strength steel. Apart from carbon by adding small % of manganese, S, P, chromium, Ni, & Cu special properties can be impact to the iron & variety of steel can be produced.

Advantages & Disadvantages Of Steel Structure.

Advantages:-

High strength

Structural steel has high strength per unit weight due to this steel member are slender when compared to the RC member.

Easy Transport ⇒

Because of small size & less weight steel members are easily transported.

* Easy to fabricate, correction & replacement -
* Highly elastic & ductile & uniformity
* Easy to strengthen the existing sub structures.
* Steel structures are gas & water tight
* Easy inspection & maintainance.
* Longer life & good scrap value.
* Material is reuseable.

Disadvantages :-

* High cost of construction.
* High maintainance cost.
* Poor fire proofing.
* They looses the strength at high temperature
* Corrossion problem.
* It requires electricity for fabrication & errection of members.
* Steel requires a skilled Laboures for errection.

Common Steel Structures.

Steel has high strength per unit weight hence it is used in construction of large common free structures. Following are the common steel structures used.

* Roof structures for factories, cinema house, auditorium etc....
* Roof trusses & columns to cover platform in railway stations & bus stands
* Transmission towers for microwave & electric power

* Water tanks, gas tankers etc...
* Chimneys.

Properties of Structural Steel [IS 800-2007 Pg 12]

Properties of steel required for engg design may be classified as.

a) Physical Properties.
b) Mechanical Properties

a) Physical Properties : P.P of structural steel irrespective of its grade may be taken as

1. Unit mass of steel, $\rho = 7850\ kg/m^3$
2. Modulus of Elasticity, $E = 2 \times 10^5\ N/mm^2$ (MPa)
3. Poission Ratio, $\mu = 0.3$
4. Modulus of Rigidity, $G = 0.76 \times 10^5\ N/mm^2$
5. Co-efficient of thermal expansion $\alpha_t = 12 \times 10^{-6}/°C$

b) Mechanical Properties IS - 800 Pg.-14

The mechanical properties of structural steel is imp in design for.

1. Yield strength (f_y)
2. The tensile or ultimate stress (f_u)
3. Max percent elongation on standarad gauge length
4. Notch toughness.

Except for notch toughness the other properties are dete-rmined by conducting tensile stress on sample cut from plates, sections etc. in accordance with IS 1608

Commoly used properties for the common structural properties of different specifications is given in fig Pg 14 - Table-1. W - Weldable.

SL.No	Indian standards	Grade	Properties fy <20	>40	fu	% of elongation	Prop fy 20-40
	ES 2062	E 165 (Fe290)	165	165	290	23	165
		E 250 (Fe 410W) A	250	230	410	23	240
		E 250 (Fe 410W) B	250	230	410	23	240
		E 250 (Fe 410W) C	250	230	410	23	240
		E 300 (Fe 440)	300	280	440	22	290
		E 390 (Fe 490)	350	320	490	20	330
		E 450 (Fe 570) D	450	420	570	20	430
		E 410 (Fe 540)	410	380	540	20	390
		E 450 (Fe 590) E	450	420	570	20	430

Rolled steel Sections.

Steel sections of standard size, shapes & length are rolled in steel mills. Various types of rolled steel sections are as follows

1. Rooled I - sections [Beams]
2. Rooled Channel sections [C - sections]
3. Rooled steel angle section [L - sections]
4. Rooled steel T section]
5. Rooled steel bars.
6. Rooled steel tubes
7. Rooled steel plates
8. Rooled steel flats
9. Rooled steel sheets & strips.

1. Rooled steel I - sections.

The following 5 - series of rooled steels are manufactured in India

a) Indian standard junior beams [ISJB]

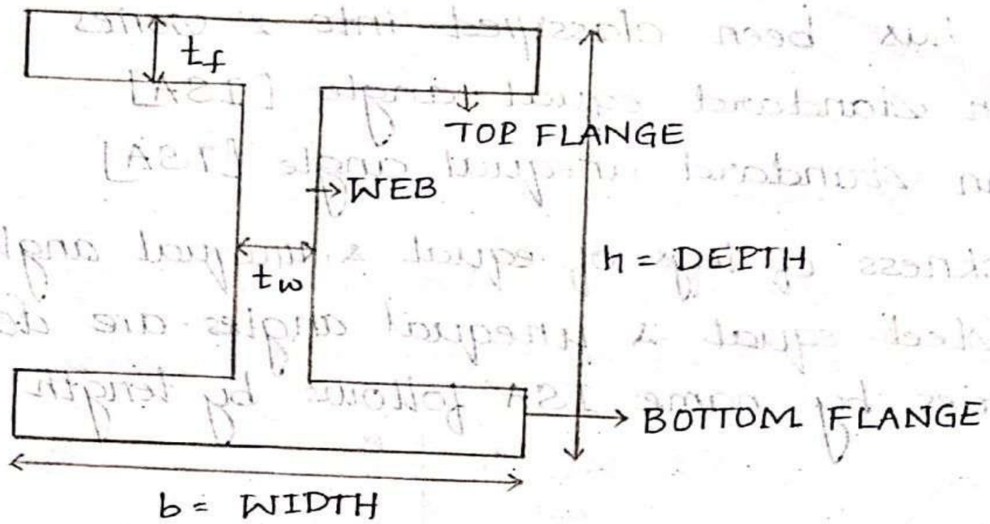
b) Indian standard Light beams [ISLB]

c) Indian standard medium beams [ISMB]

d) Indian standard medi Heavy beams [ISHB] e] Indian Standard Wide flanged beams [ISWB]

Indian Standard Light Beams.

Typical section of I - Section.



The steel sections are designated by the series to which they belong followed by depth in mm & weight per meter Length.

Example :- ISMB 500 @ 0.852 KN/m

ISMB → Series or name of section

500 → Depth in mm

0.852 KN/m → Wt/m

2] Rooled steel Channel Section.

It has been classified into 4 types.

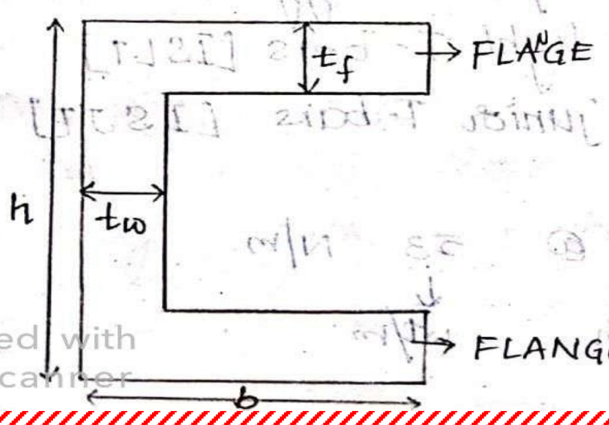
a] Indian standard junior channel [ISJC]

b] Indian standard Light channel [ISLC]

c] Indian standard medium channel [ISMC]

d] Indian standard special channel [ISSC]

Typical c/s channel is as below



Example :- ISMC 300 @ 0.351 KN/m

↓ Series Name ↓ Depth in mm ↓ wt/m

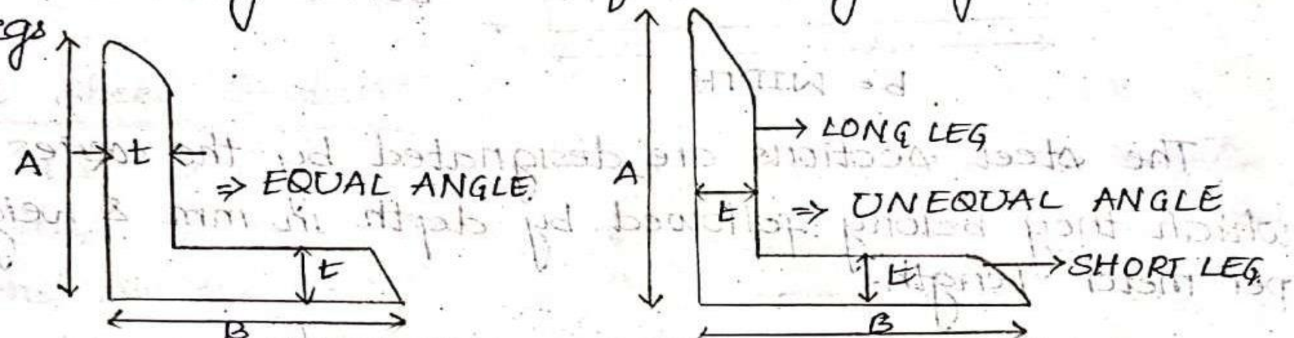
3) Rooled Steel Angle Section.

This has been classified into 2 series

a) Indian standard equal angle [ISA]

b) Indian standard unequal angle [ISA]

Thickness of legs of equal & unequal angles are same. Rooled steel equal & unequal angles are designated by their series by name ISA follows by length & thickness of legs.



Example :- ISA 150 × 150 × 10 ⇒ equal angle

↓ A ↓ B ↓ t

ISA 150 × 115 × 10 ⇒ Unequal angle

↓ A ↓ B ↓ t

4) Rooled Steel T-Angle-Section

Following are the 5-series of rooled steel sections available.

a) Indian standard normal T-section [ISNT]

b) Indian standard heavy flanged T-section [ISHT]

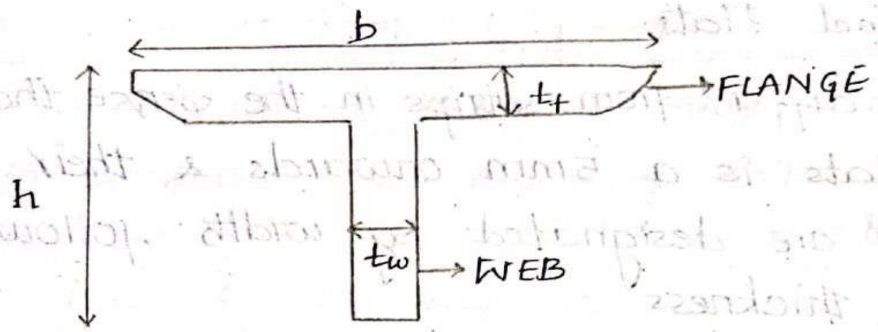
c) Indian standard special legged T-bars [ISLT]

d) Indian standard light T-bars [ISLT]

e) Indian standard junior T-bars [ISJT]

Example :- ISNT 60 @ 53 N/m

↓ Series ↓ depth ↓ wt/m

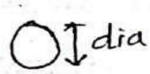


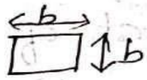
5. Rooled Steel bars.

Rooled steel bars are classified into 2 series

a) Indian standard round bars [ISRB]
b) Indian standard square bars [ISSQ]

Ex:- ISRO 20 ISSQ 10
↓ ↓
Series dia





6. Rooled Steel tubes.

This sections are designated by normal bore sides

In each size there are 3 classes namely : Light, medium & heavy

Ex:- 40 mm tube has 3 types
Light, medium & heavy.

7. Rooled Steel Plates.

This are available from 5mm - 80mm - thickness

This plates are designated by ISPL followed by length width & thickness

Example: ISPL 2000 x 1000 x 16
↓ ↓ ↓
Length Breadth Thickness

8. Rooled Steel Strips.

Rooled steel strips are designated as ISST followed by width & thickness

Example:- ISST 250 x 2.5mm
Width thickness

9. Rooled Steel Flats

Flats differ from strips in the sense that the thickness of flats is a 5mm onwards & their width is limited. Flats are designated by width followed by letter ISF & thickness

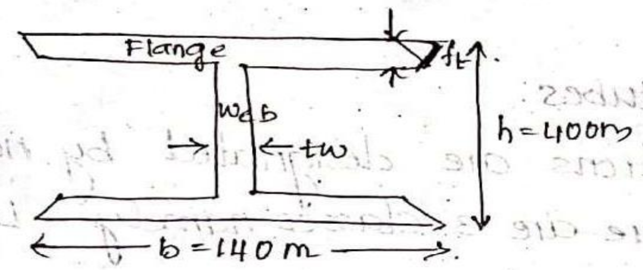
Example : 80 ISF 10

80 ↓ Width

10 ↓ Thickness

The nominal dimension, w/mtr length & all geometric Pro for various steel structure are given in sp.6 or in steel table.

Ex: Draw a neat sketch of ISMB-400 & mention its properties.



ISMB-400

Geometrical properties of above beam is as follows.

Weight = 61.6 kg/m or 604.298 KN/m

Sectional area = 7846 mm^2

Depth of section = h = 400mm

Width of flange, b = 140mm

Thickness of flange, t_f = 16mm

Thickness of Web, = t_w = 8.99 mm

Moment of inertia, $I_{xx} = 20458.4 \times 10^4 \, mm^4$

$I_{yy} = 422.1 \times 10^4 \, mm^4$

Radius of Gyration, $r_{xx} = 161.5 \, mm$

$r_{yy} = 28.2 \, mm$

Modulus of section, $Z_{xx} = 1022.99 \, mm^3$

$Z_{yy} = 89.9 \, mm^3$

Types Of Loads or Forces (Pg - 15)

For the purpose of designing any structural elements members or structure, the following loads or actions are their effects shall be taken into account where applica -ble with psf & combinations

a] Dead Loads.

b] Imposed loads (Live load, crane load, snow load, dust load, wave load, earth pressures, etc...)

c] Wind loads.

d] Earthquake loads.

e] Erection loads.

f] Accidental loads such as those due to blast, impact of vehicles. etc. &

g] Secondary effects due to contraction or expansion resulting from temperature changes.

Load Combinations [Page - 16]

Load combinations for design purpose shall be those produce maximum forces & effects consequently max stress & deformation. The following combination of load with appropriate Psf. can be considered

a] Dead Load + imposed load.

b] Dead Load + imposed load + Wind or earthquake load

c] Dead load + Wind or earthquake load

d] Dead load + erection load.

In addition to above the stresses developed due to secondary effects such as handling, erection, temperature & settlement of foundation if any shall be appropriate -ly added to the stress from the combination of loads as above.

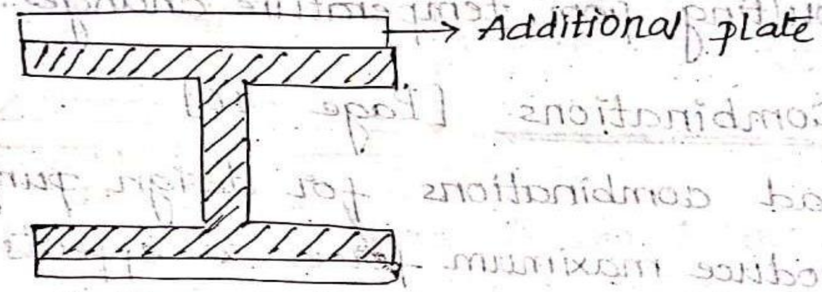
Special Consideration in Steel Design

The following special consideration are required in the steel design.

1. Size & shape.
2. Buckling.
3. Minimum Thickness.
4. Connection Designs.

1. Size & Shape :- Steel is manufactured in steel mills & is available in different shapes & sizes. Hence the member of steel structure should be designated by combination of any of the following available section.

Example :- A beam section may be standard I section or it may consists of built up section as shown in figure.



Built up section.

Sometimes the choice of the section of a member is governed by shape of the other & type of joint b/w the 2 members.

2. Buckling Consideration: The permissible load per unit area in steel is much higher as compare to the permissible value in concrete. for same load the c/s of steel is smaller. As the member in the steel str are more slender & ∴ the compression member in steel structure are liable for buckling. In case of beams there are chances of lateral buckling which creates special problems.

To account for buckling, code specifies that part of the section to be taken as ineffective.

3. Minimum Thickness.

Corrosion is special consideration in steel design if very thin sections are used a small amount of corrosion result into large % of reduction in effective area hence design practice specifies min thickness should be used in structural members.

For members directly exposed to weather the following min thickness should be used.

a) If fully accessable for cleaning painting - 6mm

b) If not accessable for cleaning painting - 8mm

4) Need for design of connections

A steel design is not complete if following connection is not designed

a) Connection b/w various standard sections selected for member.

b) Connection b/w various member like beams, column, foundation etc.... of structure.

Following 3 types of connections are commonly used.

1] Rivited connection

2] Bolted connection

3] Welded connection.

Structural Analysis - Page no 22.

Design - Pag 27 & 28

failure criteria of steel

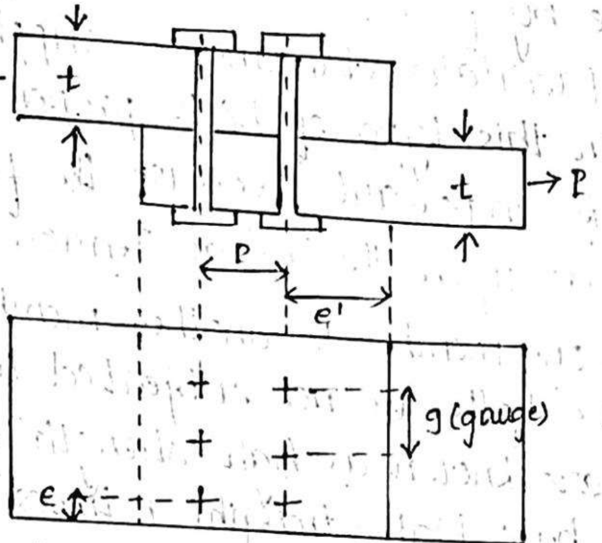
IS - code provision & specifications

selection classification (or) classification of cross-section

Pg 17

MODULE 02 - BOLTED CONNECTION

TERMINOLOGY RELEATED TO BOLTED CONNECTION
(Page No.: 1 to 5 in IS 800 : 2007)



P = pitch
g = gauge
e' = end distance
e = edge distance

mp

1) Pitch of the Bolts (P)

The centre to centre distance b/w the individual bolts in a line, in the direction of the load is called pitch

2) Gauge Distance [g]

The spacing b/w adjacent parallel lines of bolts, perpendicular to the directions of load.

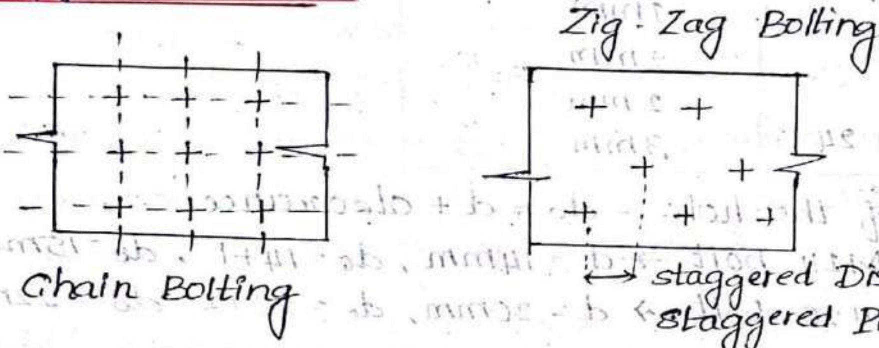
3) Edge Distance [e]

Distance from the centre of a bolt holes to the nearest edge of a plate measured perpendicular direction of the load.

4) End Distance [e']

Distance from the centre of the bolt hole to the edge of the plate measured parallel to the direction of load.

5] Staggered Distance [S] :



The distance b/w the centre of any two adjacent bolts in zig zag bolting measured parallel to the direction of the load is called staggered pitch (or) staggered distance denoted by "S"

6] Property Class Of Bolts

Bolts are grouped under different grades depending upon their strength is called property class for ex: property class 4.6 indicate the nominal ultimate tensile strength [f_u] is 400 N/mm² & nominal yield strength [f_y] is 240N/mm²

7] Specification for p, e, e', g for bolts are per IS 800 2007 Page no 73 & 74.

1] Minimum Pitch [P] = $P = 2.5d$
d = diameter of bolt

2] Maximum Pitch, $P = 16t$ (or) 200 mm for tension member
(Take the least value among those)
$P = 12t$ (or) 200mm for compression member
(take least value which ever is less)

3] Gauge Distance (g) ; $g = (100 + 4t)$ or 200mm

4] Minimum Edge Distance $e = 1.7 d_o$ d_o = dia of bolt hole for hand flame cut bolts

$e = 1.5 d_o$ → for rolled, machine flate cut, sawn & planed edges d_o = dia of hole.

5] Maximum edge distance [e] = $e = 16t$

Hole Allowance (Table 19 Page no 73)

Size of Bolt [d]	Clearance (mm)
12 - 14mm	1mm
16 - 22mm	2mm
24mm	2mm
Larger than 24	3mm

∴ diameter of the hole = $d_0 = d +$ clearance

for example M14 bolt → $d = 14mm$, $d_0 = 14+1$, $d_0 = 15mm$

M20 bolt → $d = 20mm$, $d_0 = 20+2$ $d_0 = 22mm$

for M24 bolts $d = 24mm$. $d_0 = 24+2$ ∴ $d_0 = 26mm$.

Partial safety factor

The safety of the structure depends upon 2 factor i.e., load & material strength which are not the fu? of each other and hence 2 different factors one for load & other for material strength are used, because each of two contribute partially to safety & they are termed as partial safety factor

PSF allow for uncertainity of element behaviour & possible strength reduction due to wrong functioning tolerance & imperfection in the material.

PSF for loads [Page no 29] (γ_L)

The PSF allows the possible deviation of the load, reduced possiblity of all loads acting together. Inaccurate assesstment of loads & uncertainity on effects of Loads.

The PSF for loads is load factor which is multiplied to characteristics loads & which gives the design loads.

Table 4 of IS 800 - 2007 gives PSF for loads & limitstate b] PSF for materials strength [γ_m] [Page no. 30]

The PSF for material strength allows for uncertainity of element behaviour & probability of strength reduction due to fabrication & tolerance. variation of member size. uncertainity is calculation of strength & imperfection in material. The design strength is obtained by dividing the PSF of a material.

Table 5 IS 800 - 2007 gives the PSF for materials

Types Of Bolts [In terms of Grade] : (Page no. 13, Table 1)

1] Black bolts (or) common bolts (or) Rough bolts

Grades	f_u N/mm² or MPa (ultimate Tensile stress)
3.6	330
4.6	400
4.8	420
5.6	500
5.8	520
6.8	600

2] HSFG Bolts :

Grade	f_u (N/mm² or MPa)
8.8 ($d \leq 16$mm)	800
8.8 ($d > 16$mm)	830
9.8	900
10.9	1040
12.9	1220

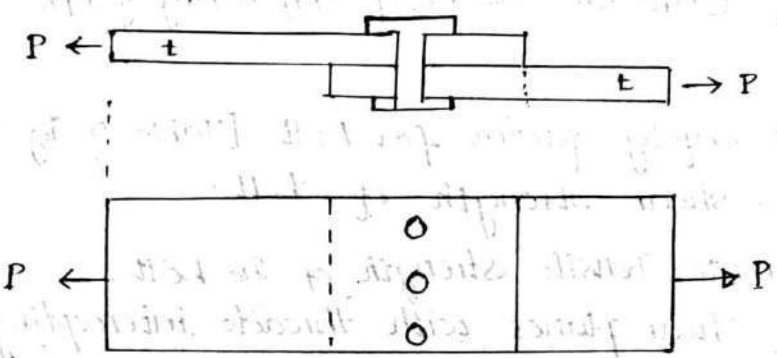
Types Of Bolted Connections.

The types of joints may be grouped into 2

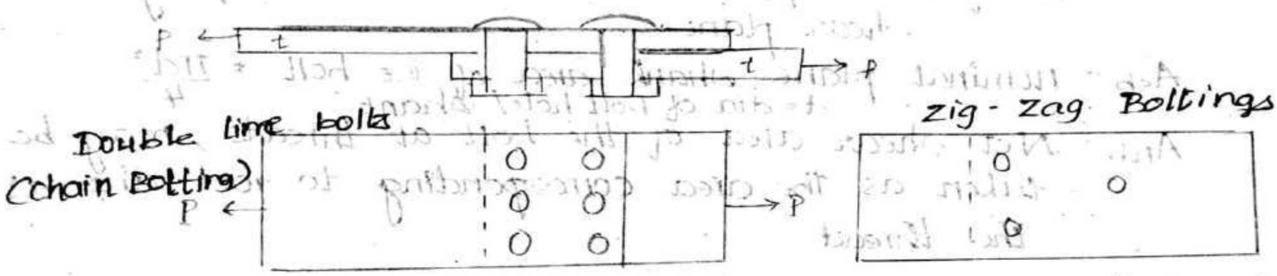
1] Lap Joint

2] Butt Joint.

1] Lap Joint.



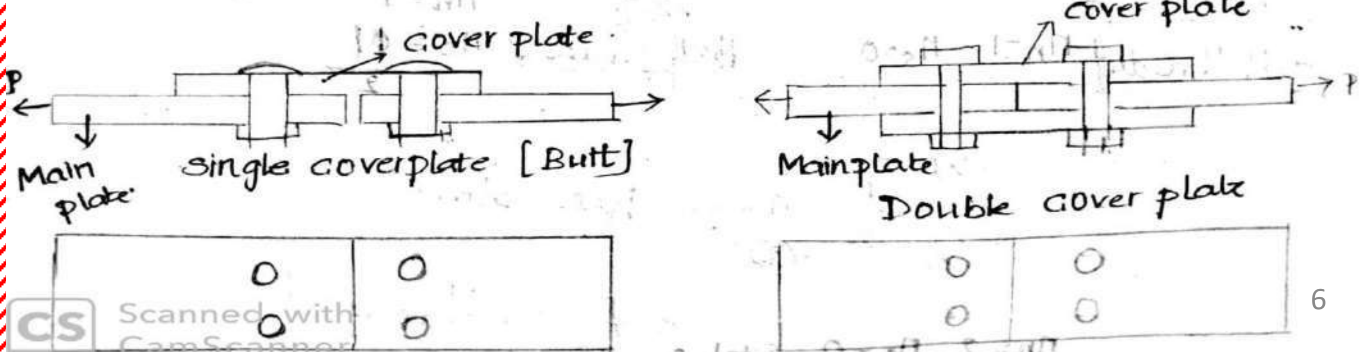
Single line Bolt



Lap joint is a simplest type of joint, in these the plates should be connected overlapped one-another. The above figure shows typical lap joints

2] Butt Joint

In this type of joint or connection 2 main plates are connected by providing a single cover plate or double cover plate, one on either side



Design Shear strength of bearing type bolts [P-75]

1) Shear capacity of the bolt [Pg. 75]

The design shear strength of the bolt is calculated by using the formula

$$\boxed{V_{dsb} = \frac{1}{\gamma_{mb}}\left[\frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)\right] \times \beta_{lj} \times \beta_{lg} \times \beta_{pk}}$$

where,
γ_{mb} = Partial safety factor for bolt [Table 5, Pg. 30]

V_{dsb} = Design shear strength of bolt.

f_{ub}/f_u = Ultimate Tensile strength of the bolt.

n_n = No of shear planes with threads intercepting the shear plane

n_s = No of shear planes without threads intercepting the shear plane.

A_{sb} = nominal plane shank area of the bolt $= \frac{\pi d^2}{4}$
d = dia of bolt hole / shank

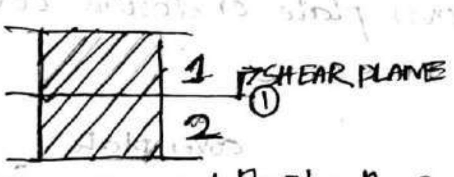
A_{nb} = Net shear area of the bolt at threads, may be taken as the area corresponding to root dia of the thread

$$A_{nb} = 0.78 \times \frac{\pi d^2}{4}$$

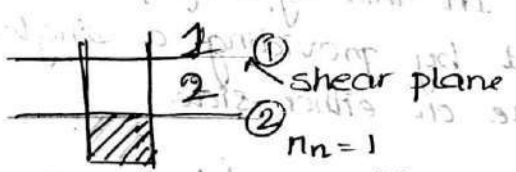
β_{lj} = Reduction for long joint

β_{lg} = Reduction for large grip length

β_{pk} = Reduction for packing plates



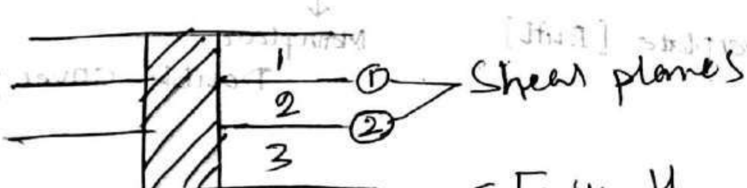
Full threading $n_n = 1,\ n_s = 0$



$n_n = 1$

Half threading $n_s = 1$

$\overline{\quad 2 \quad}$



$n_n = 2,\ n_s = 0$ Total 2 ← Fully threaded

2] Bearing Capacity / Strength of the bolt. [V_{dpb}] [Pg. 75]

The design bearing strength of bolt is calculated by following formula

$$\boxed{V_{dpb} = \frac{1}{\gamma_{mb}} [2.5\, k_b\, d\, t\, f_u]}$$

where k_b is smaller of the following.

i] $k_b = \frac{e}{3d_0}$

ii] $k_b = \frac{P}{3d_0} - 0.25$

iii] $k_b = \frac{f_{ub}}{f_u}$ $\qquad\qquad e = 1.75($

iv] $k_b = 1$

e = end distance $= 1.7 d_0$

P = pitch distance $= 2.5 d$

d_0 = Diameter of the bolt hole

f_{ub} = Ultimate tensile stress of bolt

f_u = Ultimate tensile stress of plate.

t = Least thickness of the plate.

∴ bolt value is least of V_{dsb} & V_{dpb}

3] Shear Strength of HSFG bolts [Pg 76]

$$\boxed{V_{dsf} = \frac{1}{\gamma_{mf}} [\mu_f\, n_e\, K_h\, F_o]}$$

where μ_f = co-efficient of friction (slip factor) ($\mu_f = 0.55$) [Tb. 20]

n_e = no of effective interfaces offering frictional resistance to slip

K_h = 1.0 for fasteners in clearance holes.

= 0.85 for fasteners in oversized & short slotted holes.

= 0.7 for fasteners in long slotted holes.

γ_{mf} = 1.10 or 1.25 depends on slip resistance

F_0 = minimum bolt tension = $A_{nb} f_0$

A_{nb} = net area of the bolt at threads $= \frac{\pi d^2}{4} \times 0.78$

f_0 = proof stress = $0.70 f_{ub}$

$\therefore \quad F_0 = 0.78 \frac{\pi d^2}{4} \times 0.70 f_{ub}$

The bolt value of HSFG, V_{dsf}

1) Black bolts ⇒ BV = V_{dsb} & V_{dbp}

2) HSFG bolts ⇒ BV = V_{dsf}

Design tensile strength of plates in joints [Pg no 32]

The plates in a joint made with bearing bolts may fail under tensile force due to any one of the following

1) Brusting / Tearing of Edges.

2) Crushing of plates

3) Rupture of plates.

Design strength due to yielding of Gross section [Pg. 32]

The design strength of members under axial tension T_{dg}, as governed by yielding of gross-section is given

$$\boxed{T_{dg} = \frac{A_g f_y}{\gamma_{mo}}}$$

where f_y = Yield stress of the material

A_g = gross area of the c/s

γ_{mo} = Partial safety factor for failure in tension by yielding (Table 5)

Design strength due to Rupture of Critical Section

The design strength in tension of a plate T_{dn}, as governed by rupture of net cross-section area A_n at the holes is given by

$$\boxed{T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}}$$

where, γ_{m1} = Partial safety factor for failure at ultimate stress

f_u = Ultimate stress of the material

A_n = net effective area of the member given by

$$\boxed{A_n = \left[b - n d_h + \sum_i \frac{P_{si}^2}{4 g_i}\right] t}$$

where b, t = width & thickness of the plate

d_h = diameter of the bolt hole

g = Gauge length b/w bolt hole, as

P_s = staggered pitch length b/w line of bolt hole

n = no of bolt holes in the critical section

i = subscript for summation of all the inclined legs

Efficiency Of Joint

It is defined as the ratio of strength of joint & strength of plate in tension. It is usually expressed in percentage.

(least of 4 shear, bearing, rupture, yielding)

$$\therefore \text{Efficiency } (\eta) = \frac{\text{Strength of the joint}}{\text{Strength of the plate}} \times 100\ \%$$

(yielding strength)

Problems

(1) Determine strength of M16 property class 5.6 black bolts which is used to connect 10mm thick & 8mm thick plates using lap joint. Take pitch = 50mm, end distance = 30mm. The grade of plate is Fe410

Sol^n: Strength / Bolt value = ?

For M16 ⇒ $d = 16mm$

$d_0 = d + \text{Clearance}$. [from table allowance]

$= 16 + 2$

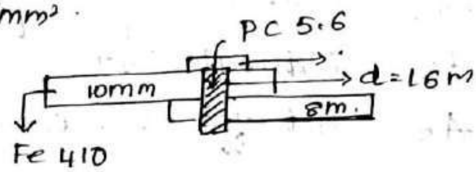
$d_0 = 18mm$

For property Glass 5.6 → $f_{ub} = 500\ N/mm^2$ (Pg. no 13)

For plate ⇒ Fe 410 = 410 N/mm² [Pg 14, E 250(Fe 410W)A]
$f_u = 410\ N/mm^2$

P = Pitch = 50 mm.

e = 30 mm



t = 8mm [least 10 & 8)

a] For black bolt

a) Shear strength $= V_{dsb} = \frac{1}{\gamma_{mb}}\left[\frac{f_{ub}}{\sqrt{3}}[n_n A_{nb} + n_s A_{sb}]\right]\times$

$\gamma_{mb} = 1.25$ [Pg no. 30]

$A_{nb} = 0.78\,\frac{\pi d^2}{4} = 0.78 \times \frac{\pi(16)^2}{4}$

$n_n = 1,\quad n_s = 0$

$A_{nb} = 156.82\,mm^2$

$$V_{dsb} = \frac{1}{1.25}\left[\frac{500}{\sqrt{3}}[1\times 156.82]\right]$$

$$\boxed{V_{dsb} = 36.216\times10^3\ N/\cancel{mm^2}}$$

$= 36.216\ KN/\cancel{mm^2}$

b] Bearing strength of bolt

$$V_{dpb} = \frac{1}{\gamma_{mb}}\left[2.5\,K_b\,d\,t\,f_u\right]$$

$K_b = \frac{P}{3d_0} - 0.25$, $\gamma_{mb} = 1.25$, $K_b = \frac{e}{3d_0} = \frac{30}{3\times18} = 0.55$

$= \frac{50}{3\times18} - 0.25$ $\quad K_b = \frac{f_{ub}}{f_u} = \frac{500}{410} = 1.21$

$= 0.675$ $\quad K_b = 1$

∴ $K_b = 0.55$, $t = 8mm$, $f_u = 410$, $d = 16mm$

$$V_{dpb} = \frac{1}{1.25}\left[2.5\times0.56\times16\times8\times410\right]$$

$$\boxed{V_{dpb} = 58.77\times10^3\ N \text{ or } 58.77\ KN}$$

∴ Strength of bolt joint or bolt value = 36.216 KN

2] Determine the bolt value for M22, G - 5.6 property class holes applied in double shear. Assume threads in shear plane. Bolts are used connect angles to 10mm thick ~~gauste~~ plate

Sol^n: For M22 bolt ⇒ d. = 22mm

d_o = 22 + clearance [Pg 73

= 22 + 2

d_o = 24 mm

For property class 5.6. → f_{ub} = 500 N/mm².

γ_{mb} = 1.25

Assume Fe 410 plate. ⇒ F_u = 410 N/mm²

thickness = t = 10mm

a) Design shear strength of bolt

$$V_{dsb} = \frac{1}{\gamma_{mb}}\left[\frac{f_{ub}}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]\right]$$

For double shear ~~Kn~~ & threads in shear plane, $n_n = 2$

$n_s = 0$

$$A_{nb} = 0.78 \times \frac{\pi (22)^2}{4} = 296.50 \text{ mm}^2$$

$$A_{sb} = \frac{\pi (22)^2}{4} = 380.13 \text{ mm}^2$$

$$V_{dsb} = \frac{1}{1.25}\left[\frac{500}{\sqrt{3}}\left[2 \times 296.5\right]\right] = 136.94 \text{ KN}$$

b) Design Bearing strength of bolt

P = 2.5d = 55

e = 1.7do = 45

$$V_{dpb} = \frac{1}{\gamma_{mb}}\left[2.5\ k_b\, d\, t\, f_u\right]$$

$$k_b = \frac{P}{3d_o} - 0.25 = \frac{2.5d}{3d_o} - 0.25 = \frac{2.5 \times 22}{3 \times 24} - 0.25 = 0.513$$

$$k_b = \frac{e}{3d_o} = \frac{1.7d_o}{3d_o} = \cancel{\frac{4.0 \times 24}{3 \times 24}} = \frac{45}{3 \times 24} = 0.62$$

$$k_b = \frac{f_{ub}}{f_u} = \frac{500}{410} = 1.21 \quad , \quad k_b = 1$$

$$V_{dpb} = \frac{1}{1.25}\left[2.5 \times 0.51 \times 22 \times 10 \times 410\right]$$

~~73.6032 KN~~

= 92.5452 KN

3] Determine the strength M_{18} property class 8.8 HSFG bolts connected to plates of 10mm thick

Sol^n. For M-18 $d = 18mm$

$d_o = 18 + 2 = 20mm$

For property class 8.8 $(d > 8.8)$ $f_{ub} = 830\ N/mm^2$ [Pg. 13]

a] Design shear strength of HSFG bolts

$$V_{dsf} = \frac{1}{\gamma_{mf}}\left[\mu_f \times n_e \times K_h \times F_o\right] \rightarrow \text{Pg 76.}$$

$\gamma_{mf} = 1.25$, $n_e = 2$, $K_h = 1$

$\mu_f = 0.55$

$F_o = A_{nb} \times f_o$

$= 0.78 \times \frac{\pi d^2}{4} \times 0.7 \times f_{ub}$

$= 0.78 \times \frac{\pi (18)^2}{4} \times 0.7 \times 830$

$= 115.32 \times 10^3\ N$

$$V_{dsf} = \frac{1}{1.25}\left[0.55 \times 2 \times 1 \times 115.32 \times 10^3\right]$$

$$\boxed{V_{dsf} = 101.48\ KN}$$

Problems On Efficiency

$$\text{Efficieny}\ (\eta) = \frac{\text{strength of joint}}{\text{strength of plate Yield}} \times 100$$

Strength of joint is taken as least of V_{dsb}, V_{dpb}, T_{dg} & T_{dn}.

1] Determine the efficiency of lap joints b/w plates 100x8mm using the black bolts of 12mm dia & grade 4.6. The plates are of steel of grade Fe 410

Sol^n: $\eta = ?$

$d = 12mm$

$d_o = 12 + 1 = 13mm$

For grade 4.6, $f_{ub} = 400\ N/mm^2$ [Pg. 13]

$\gamma_{mb} = 1.25$

$\gamma_{mf} = 1.25$

For Fe 410 plate $f_u = 410\ N/mm^2$

For fe 410. $f_y = 250 N/mm^2$ [Pg. 14]

Pitch $= P = 2.5d = 2.5 \times 12 = 30 mm$

$= e = 1.7 d_o = 1.7 \times 13 = 22.1 mm \simeq 25$ [say]

a) Total shear strength of bolt

$$V_{dsb} = \frac{1}{\gamma_{mb}} \left[\frac{f_{ub}}{\sqrt{3}} \left[n_n A_{nb} + n_s A_{sb} \right] \right]$$

Assuming a single plane & threaded bolt (fully)

$n_n = 1, \quad n_s = 0$

$$A_{nb} = 0.78 \times \frac{\pi (d^2)}{4} = 0.78 \times \frac{\pi (12)^2}{4} = 88.21 \ mm^2.$$

$$= \frac{1}{1.25} \left[\frac{400}{\sqrt{3}} \left[1 \times 88.21 \right] \right]$$

$$\boxed{V_{dsb} = 16.29 \ KN}$$

b) Bearing strength of bolt

$$V_{dpb} = \frac{1}{\gamma_{mb}} \left[2.5 \ K_b \ t \ d \ f_u \right]$$

$K_b \Rightarrow$ i) $\frac{P}{3 d_o} = \frac{30}{3 \times 13} = 0.769 - 0.25 = 0.519$

ii) $\frac{e}{3 d_o} = 0.64$

iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$

$$V_{dpd} = \frac{1}{1.25} \left[2.5 \times 0.51 \times 8 \times 12 \times 410 \right]$$

$$= 40.14 \ KN$$

c) Design Yield strength of plate (T_{dg})

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

$A_g = 100 \times 8 = 800 \ mm^2$

$f_y = 250 \ N/mm^2$

$\gamma_{mo} = 1.10$ [Pg - 30.

$$T_{dg} = \frac{800 \times 250}{1.10}$$

$$\boxed{T_{dg} = 181.81 \ KN}$$

d) Design rupture strength of plate

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}}$$ $\gamma_{mL} = 1.25$ [Pg. 30]

$$A_n = \left[b - n d_n + \sum \frac{p s_i^2}{4 g_l}\right]^{\to 0} t$$ [since it is not staggered arrange -ment

$= [b - n d_n] t$

$= [100 - 1 \times 13] \times 8$

$\boxed{A_n = 696\,mm^2}$

$T_{dn} = \frac{0.9 \times 696 \times 410}{1.25} = 205.45\,KN$

$T_{dn} = 205.4N\,KN$

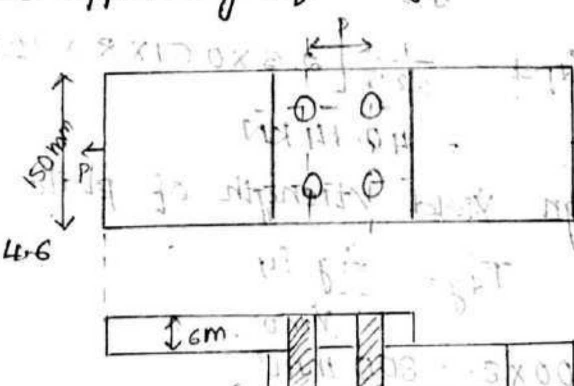
Therefore least strength of the above 4 values = 16.29 KN

$\therefore \quad \eta = \frac{\text{least strength}}{\text{Yield strength}} = \frac{16.29}{181.8} \times 100$

$\boxed{\eta = 8.96\,\%}$

---

2) 2 plates of size 150mm x 6mm are connected by a double bolted lap joint using 4 - 16mm dia unfinished bolts. Determine the efficiency of the joint.



Given :-

$d = 16mm$

$d_0 = 16+2 = 18mm$

Assume property class 4.6

$\therefore f_{ub} = 400 N/mm^2$

$\gamma_{mb} = 1.25$

Assume 410 plate, $\therefore f_u = 410\, N/mm^2$

$f_y = 250$ [Pg. 14]

Pitch, $p = 2.5d = 2.5(16) = 40mm$

$e = 1.7 d_0 = 1.7(18) = 30.6 \simeq 35mm$

$t = 6mm$

$b = 150\,mm$

a) Total shear strength of black bolt

$$V_{dsb} = n\left[\frac{1}{\gamma_{mb}}\left\{\frac{f_{ub}}{\sqrt{3}} \times [n_n A_{nb} + n_s A_{sb}]\right]\right]$$

$A_{nb} = 0.78 \times \frac{\pi d^2}{4} = 0.78 \times \frac{\pi(16)^2}{4} = 156.82\, mm^2$

For single plane & fully threaded bolts. $n_n = 1$; $n_s = 0$

$$= 4\left[\frac{1}{1.25}\right] \times \left[\frac{400}{\sqrt{3}} \times [1 \times 156.82]\right]$$

$V_{dsb} = 115.89\, KN$

$P = 2.5d = 40$
$e = 1.7 d_0 = 35$

b) Bearing strength of black bolt

$$V_{dpb} = N\left[\frac{1}{\gamma_{mb}}[2.5\, K_b\, t\, d\, f_u]\right]$$

$K_b =$ i) $\frac{P}{3d_0} - 0.25 = 0.497$

ii) $\frac{e}{3d_0} = 0.648$

iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

iv) $K_b = 1$

$$= 4\left[\frac{1}{1.25}[2.5 \times 0.5 \times 6 \times 16 \times 410]\right]$$

$V_{dpb} = 157.44\, KN$

c) Yield strength of plate

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$$

$A_g = 150 \times 6 = 900\, mm^2$
$f_y = 250$
$\gamma_{m0} = 1.10$

$$= \frac{900 \times 250}{1.10}$$

$T_{dg} = 204.54\, KN$

n = no of bolts in critical sectio

critical sed.

d) Design rupture strength of plate

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

$A_n = [b - n d_n] \times t$
$= [150 - 2 \times 18] \times 6$
$A_n = 684\, mm^2$
$\gamma_{m1} = 1.25$ [Eq. 30]

$$= \frac{0.9 \times 684 \times 410}{1.25}$$

$T_{dn} = 201.916\, KN/$~~mm~~

Strength of joint (or) max load the joint can carry $= 115.89\, KN$

$\therefore \quad \eta = \dfrac{\text{strength of least}}{\text{Yield strength}} \times 100 = \dfrac{115.89}{204.54} \times 100$

$$\boxed{\eta = 56.65\%}$$

3) Determine nominal shear capacity, design shear strength, nominal bearing strength & design strength in bearing for M16, property class 8.8 bolts. assuming bolt threads outside the shear plane. Bolts are connected to 12mm thick plate. Assume end distance of bolt = 30mm, pitch = 80mm, f_u = 410 MPa, A_{sb} = 201 mm²

Sol^n: Data: $V_{nsb} = ?$ For M16 $d = 16$ mm

$V_{dsb} = ?$ $d_o = 16 + 2 = 18$ mm

$V_{npb} = ?$ For property class 8.8

$V_{dpb} = ?$ $f_{ub} = 800 \text{ N/mm}^2$ [13]

$t = 12$ mm $\gamma_{mb} = 1.25$ [Pg. 30]

$e = 30$ mm

$P = 80$ mm

$f_u = 410 \text{ N/mm}^2$

$A_{sb} = 201 \text{ mm}^2$

① Nominial Shear strength

$$V_{nsb} = \frac{f_u}{\sqrt{3}}\left[\cancel{n_n A_{nb}}^{\,0} + n_s A_{sb}\right] \qquad n_n = 0,\ n_s = 1$$

$$= \frac{410}{\sqrt{3}}\,[1 \times 201]$$ since bolt threads outside the shear plane.

$$= 47.57 \text{ KN}$$

2) Design shear strength (75)

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{47.57}{1.25}$$

$$V_{dsb} = 38.056 \text{ KN}$$

3) Nominal Bearing strength

$$V_{npb} = [2.5 \cdot K_b\, d\, t\, f_u]$$

$$= [2.5 \times 0.55 \times 16 \times 12 \times 410]$$

$$V_{npb} = 108.24 \text{ KN}$$

$K_b = \dfrac{e}{3d_o} = 0.55$

$K_b = \dfrac{P}{3d_o} - 0.25 = 1.23$

$K_b = \dfrac{f_u}{f_{ub}} = 0.97$

$K_b = 1$

4] Design of strength in bearing

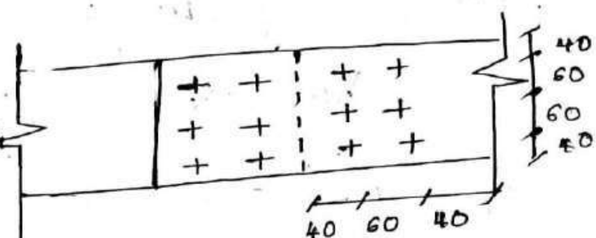
$$V_{dpb} = \frac{1}{\gamma_{mb}} \left[2.5 K_b t d f_u \right]$$

$$= \frac{1}{1.25} \times 2.5 \times 0.55 \times 12 \times 16 \times 410$$

$$\boxed{V_{dpb} = 86.59 \text{ KN}}$$

4] 2 plates 10mm & 18mm thick are to be joined by a double cover butt joint. Assuming cover plates of 6mm thickness. Evaluate the joint strength & calculate it efficiency. using M20 bolts of grade 4.6 & fe 410 plate. Assume a pitch of 60mm & edge distance of 40mm.

Sol^n: Data :-



All dim are in mm.

For M20, $d = 20$mm

$d_0 = 20 + 2 = 22$mm

For grade 4.6

$f_{ub} = 400 \text{ N/mm}^2$

$\gamma_{mb} = 1.25$

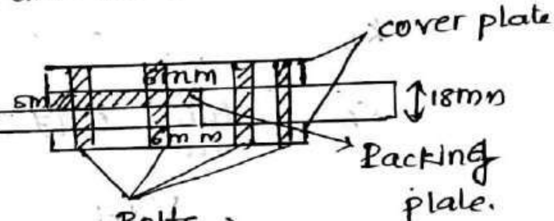
For Fe 410 plate

$f_u = 410 \text{ N/mm}^2$

$f_y = 250 \text{ N/mm}^2$ [Pg. 14]

$P = 60$mm

$e = 40$mm.



Sum of thickness of cover plates = 12mm

Least thickness of plate = 10mm

∴ On comparision b/w these 2 value the least value among this = 10mm.

N = 6. [no of bolts on one side of the joint]

a) Total shear strength of the plate.

$$V_{dsb} = N \times \frac{1}{\gamma_{mb}} \left[\frac{f_{ub}}{\sqrt{3}} \times [n_n A_{nb} + n_s A_{sb}] \right] \times \beta_{pk}$$

$$= 6 \times \frac{1}{1.25} \left[\frac{400}{\sqrt{3}} \times [1 \times \right.$$

Assume one plane is intercepting the shank & other plane intercepting the thread ∴ $n_n = 1$; $n_s = 1$

where β_{pk} = reduction factor for packing plate

$\beta_{pk} = [1 - 0.0125\, t_{pk}]$ [Pg. no 75]

t_{pk} = Thickness of the packing plate = 8mm.

$\beta_{pk} = [1 - 0.0125 \times 8]$

$= 0.9$

$A_{nb} = \frac{\pi d^2}{4} \times 6 = 1884.94$

$A_{sb} = 0.78 \times \frac{\pi d^2}{4} = 1470.2$

$V_{dsb} = 6 \times \frac{1}{1.25}\left[\frac{400}{\sqrt{3}} \times 1 \times 314.15\right]$

$=$

$= \cancel{47.89 kN}$ $\boxed{V_{dsb} = 557.89 kN}$

b] Total Bearing Strength.

$V_{dpb} = N \times \frac{1}{\gamma_{mb}}[2.5 \times k_b \times t \times d \times f_u]$

$k_b = \frac{p}{3d_o} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.65$

$k_b = 1$

$k_b = \frac{e}{3d_o} = 0.60$

$k_b = \frac{f_{ub}}{f_u} = 0.97$

$V_{dpb} = 6 \times \frac{1}{1.25}[2.5 \times 0.60 \times 10 \times 20 \times 410]$

$\boxed{V_{dpb} = 600.24 kN}$

c] Design Yield Strength

$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$

$f_y = 250 N/mm^2$

$b = 40 + 40 + 60 + 60 = 200$

$A_g = b \times d = 200 \times 10 = 2000 mm^2$

$\gamma_{mo} = 1.10$

$T_{dg} = \frac{2000 \times 250}{1.10}$

$\boxed{T_{dg} = 454.54 kN}$

d] Design Rupture Strength of the plate

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

$$A_n = [b - n d_n] t + \cancel{\Sigma \frac{P_{si}^2}{4g_i} \times t} \; 0$$

$$= [200 - 3 \times 22] \times 10$$

$$A_n = 1340 mm^2$$

$$= \frac{0.9 \times 1340 \times 410}{1.25}$$

$$\boxed{T_{dn} = 395.57 \, KN}$$

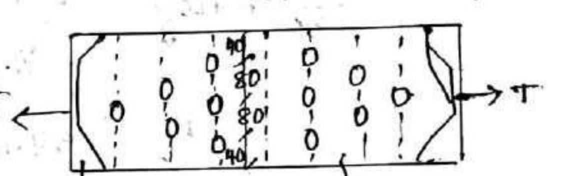
∴ strength of the joint is the least of 4 values = 395.57

$$\therefore \text{Efficiency} = \frac{\text{Least Strength}}{\text{Yield strength}} \times 100$$

$$= \frac{395.57}{454.54} \times 100$$

$$\boxed{\eta = 87.30\%}$$

5] Determine the strength of butt joint shown in figure. Use M20 property class 8.8 HSFG bolts. also find efficiency of the joint.



Sol^n: For M20, $d = 20mm$

$d_o = 20 + 2 = 22mm$

For property class 8.8

$f_{ub} = 830 \, N/mm^2$

Assume grade of plate as, Fe 410

$f_y = 250 \, N/mm^2$

$\gamma_{mb} = 1.25$, $\gamma_{mo} = 1.10$

∴ $t = 6 + 6 = 12mm$ & $10mm$. (take the Least)

$t = 10mm$

a] Shear strength of HSFG bolt

$$V_{dsf} = \frac{N \cdot 1}{\gamma_{mf}} [\mu_f \, n_e \, K_n \, F_0]$$

$$= 6 \frac{1}{1.10} [0.55 \times 2 \times 1 \times 142.37]$$

$$= 854.22 \, KN$$

$\gamma_{mf} = 1.10$

$n_e = 2$ [2 planes)

$K_n = 1$

$\mu_f = 0.55$

$F_0 = 0.7 \, f_{ub} \, A_{nb}$

$= 0.7 \times 830 \times 0.78 \times \frac{\pi (20)^2}{4}$

$F_0 = 142.37 \times 10^3 N$

b] Design Yield strength [Pg. 32]

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{240 \times 10 \times 250}{1.10}$$

$$\boxed{T_{dg} = 545\ KN}$$

$b = 240\ mm$

$A_g = b \times d = 240 \times 10$

$f_u = 410$

$\gamma_{m1} = 1.125$

c] Design Rupture strength

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

$A_{n_{1-1}} = [b - n d_n] t = [240 - 1(22)] 10 = 2180\ mm^2$

$$T_{dn_{1-1}} = \frac{0.9 \times 2180 \times 410}{1.125} = \cancel{715.04\ KN}\ 643.53\ KN$$

$A_{n_{2-2}} = [240 - 2(22)] 10 = 1960\ mm^2$

$$T_{dn_{2-2}} = \frac{0.9 \times 1960 \times 410}{1.25} = 578\ KN\text{-}m$$

$A_{n_{3-3}} = [240 - 3(22)] 10 = 1740\ mm^2$

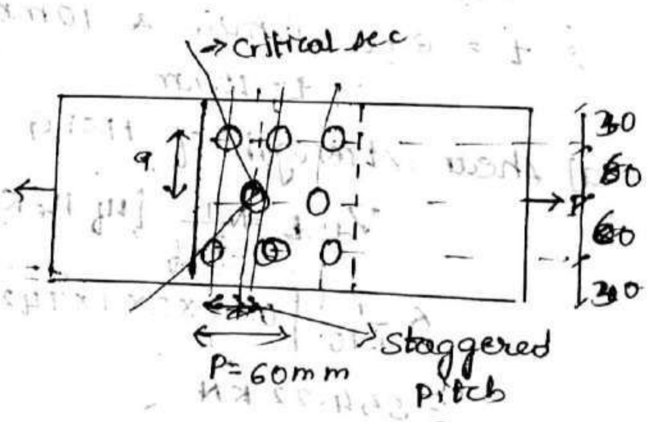
$$T_{dn_{3-3}} = \frac{0.9 \times 1740 \times 410}{1.25} = 513.6\ KN\text{-}m$$

∴ strength of joint = 513.64 KN (least of above all values)

$$\therefore \text{Efficiency} = \frac{\text{least strength}}{\text{Yield strength}} = \frac{513.64}{545.45} \times 100$$

$$\boxed{\eta = 94.16\ \%}$$

6] Determine strength of the joint & its efficiency when two plates of 8mm each is connect with zig-zag bolts as shown in figure. Use M16 bolts & property class 6.8.



Sol^n: $d = 16\ mm$

$d_0 = 16 + 2 = 18\ mm$

For property class 6.8

$f_{ub} = 600\ N/mm^2$

Assume Grade of plate = 410

$f_u = 410$

$p = 60mm$, $P_s = 30mm$

$g = 80mm$, $t = 8mm$. $b = 240 mm$, $f_y = 250 N/mm^2$, $\gamma_{mb} = 1.25$

a] Total shear strength

$$V_{dsb} = N \times \left[\frac{1}{\gamma_{mb}} \left[\frac{f_{ub}}{\sqrt{3}} \left[n_n A_{nb} + n_s A_{sb} \right] \right] \right]$$

Assume shear planes in thread. $n_n = 1$, $n_s = 0$.

$$= 8 \times \left[\frac{1}{1.25} \times \left[\frac{600}{\sqrt{3}} \left[1 \times 0.78 \times \pi \frac{(16)^2}{4} \right] \right] \right]$$

$$\boxed{V_{dsb} = 347.7 \ KN}$$

b] Total bearing strength

$$T_{dpb} = N \left[\frac{1}{\gamma_{mb}} (2.5 K_b \, t \, d \, f_u) \right]$$

$$K_b = \frac{P}{3d_0} = \frac{60}{3 \times 18} = \qquad K_b = \frac{e}{3d_0} =$$

$$= 8 \left[\frac{1}{1.25} (2.5 \times 0.74 \times 18 \times 8 \times 410) \right]$$

$$\boxed{T_{dpb} = 621.26 \ KN}$$

c] Design Yield strength [Eq. 32]

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{(240 \times 8) \times 250}{1.10}$$

$$\boxed{T_{dg} = 436.36 \ KN}$$

d] Design of Rupture strength $\gamma_{mL} = 1.25$

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{mL}}$$

$$A_n = \left[[b - n d_n] + \left[\Sigma \frac{P_{si}^2}{4 g_i} \right] \right] t$$

~~{240 - (3 × 18)} +~~

m = no of zig zag lines

$$= \left[\{b - n d_n\} + \frac{m P_s^2}{4 g} \right] \times t$$

$$= \left[240 - (3 \times 18) + \frac{2 \times (30)^2}{4 \times 80} \right] \times 8$$

$$\boxed{A_n = 1533 \ mm^2}$$

$$= \frac{0.9 \times 1533 \times 410}{1.25}$$

$$T_{dn} = \cancel{452.89\ KN} \quad 452.54$$

$\therefore$ Strength of joint $= 347.75$ KN (Least of the 4 values)

$\therefore$ efficiency $= \dfrac{\text{Least strength}}{\text{Yield strength}} \times 100$

$$= \frac{347.75}{436.65} \times 100$$

$$\eta = 79.69\ \%$$

Design Of Joints

Following procedure is adopted to design the joint [Lap or Butt Joint]

1] Find Bolt Value.

2] Find no of bolts. ⇒ $N = \frac{\text{Factored load}}{\text{Bolt Value}}$

3] Arrangement of bolts.

4] Check for strength due to rupture of critical section of the plate i.e., $T_{dn} >$ Factored Load.

---

① Design a bolted connection for a lap joint of plate thickness 10mm & 12mm to carry a service load of 100KN. Use M16 4.6 grade bolts. Give the details with sketch [Assume the bolts as fully threaded.

Sol^n: Data:

$t_1 = 10mm$, $t_2 = 12mm$

$t = 10mm$

service load = 100 KN

∴ Factored load = $1.5 \times 100 = 150$ KN

For M 16, $d = 16mm$

$d_o = 16 + 2 = 18mm$

For 4.6 grade bolt - $f_{ub} = 400 \, N/mm^2$

~~$f_y = 250 \, N/mm^2$~~

$\gamma_{mb} = 1.25$; $\gamma_{mo} = 1.10$

Assume Fe 410 plate

$f_u = 410\ N/mm^2$, $f_y = 250\ N/mm^2$

i] Shear Strength of black bolt.

$$V_{dsb} = \frac{1}{\gamma_{mb}}\left[\frac{f_{ub}}{\sqrt{3}} \times \left[N_n A_{nb} + \cancel{N_s A_{sb}}^{\,0}\right)\right] \qquad N_n = 1,\ N_s = 0$$

since bolts are fully threaded & in single plane.

$$= \frac{1}{1.25}\left[\frac{400}{\sqrt{3}} \times \left[1 \times 0.78 \times \pi \times \frac{(16)^2}{4}\right]\right]$$

$V_{dsb} = 28.97\ KN$

ii] Design Bearing Strength of bolt.

$$V_{dpb} = \frac{1}{\gamma_{mb}}\left[2.5 \times K_b \times d \times t \times f_u\right] \qquad p = 2.5d = 40 \qquad e = 1.7 d_o = 30.6 \simeq 35$$

$K_b = \frac{p}{3d_o} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.49$; $K_b = \frac{e}{3d_o} = 0.64$

$K_b = 1$ $\qquad K_b = \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

$\therefore K_b = 0.49$

$$= \frac{1}{1.25}\left[2.5 \times 0.49 \times 16 \times 10 \times 410\right]$$

$V_{dpb} = 64\ KN$

$\therefore$ Bolt Value $= 28.97\ KN$ [Least of the above 2]

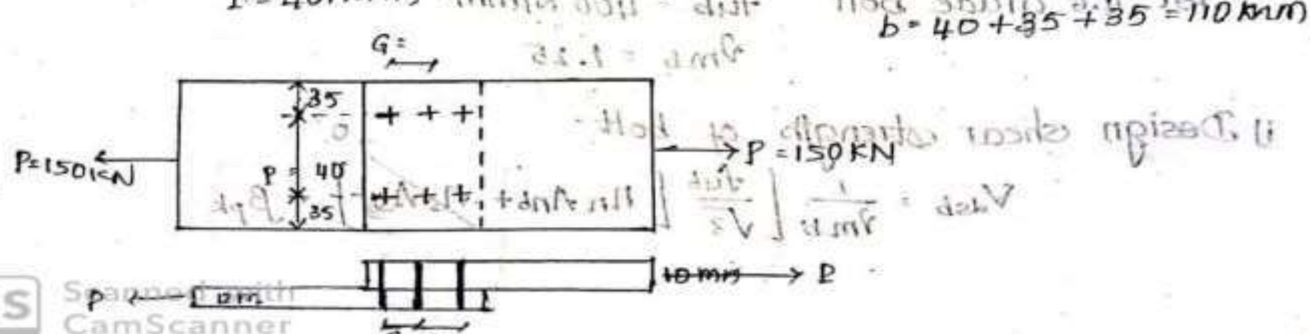
iii] No of bolts

$$N = \frac{\text{Factored load}}{\text{Bolt Value}} = \frac{150}{28.97} = 5.17 \simeq 6\ \text{no}$$

iv] Arrangement of bolts

$P = 40mm$, $e = 35mm$, $N = 6$

$b = 40 + 35 + 35 = 110\ mm$.



v) Check for strength due to rupture of critical section of plate

i.e., $T_{dn} >$ Factored load.

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}} \qquad A_n = [b - n d_n] t$$

$$= [110 - 2 \times 18] 10$$

$$= 740 mm^2$$

$$= \frac{0.9 \times 740 \times 410}{1.25}$$

$$T_{dn} = 218.4 \text{ KN}$$

$\therefore$ 218.4 > 150 KN ; Hence safe.

---

2) 2-plates 10mm & 18mm thick are to be jointed by double cover butt joint. Design the joint for the following data

i) Factored design load = 750 KN
ii) Bolt diameter = 20mm
iii) Grade of steel = Fe 410
iv) Grade of bolts = 4.6
v) Cover plates = 2 one-one each side = 8mm

Show the details by drawing a rough sketch.

Sol^n: Factored design load = 750 KN. = P

$d = 20mm, \quad d_0 = 22mm$

For butt joint

$t = 8 + 8 = 16$ mm & compare with least of 2-plate thickness i.e., 10 mm

$\therefore t = 10$ mm

For Fe 410 plate, $f_u = 410 \text{ N/mm}^2$

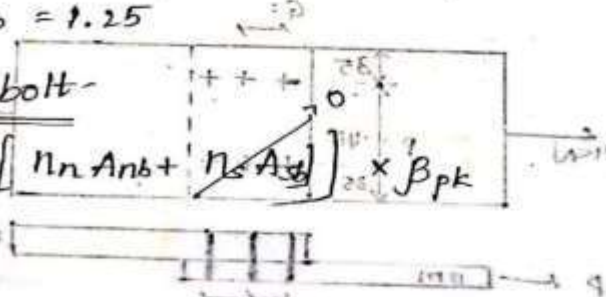
$f_y = 250 \text{ N/mm}^2$

For 4.6 grade bolt $f_{ub} = 400 \text{ N/mm}^2$

$\gamma_{mb} = 1.25$

i) Design shear strength of bolt -

$$V_{dsb} = \frac{1}{\gamma_{mb}} \left[\frac{f_{ub}}{\sqrt{3}} \left[n_n A_{nb} + \cancel{n_s A_{sb}} \right] \right] \times \beta_{pk}$$



$\beta_{pk} = [1 - 0.0125 \times t_{pk}]$

$= [1 - 0.0125 \times 8)$

$\beta_{pk} = 0.9$

Assume shear planes are intercepting the threaded area.

Assume fully threaded bolt & 2 planes are intercepting the threads.

$\therefore n_n = 2, \quad n_s = 0, \quad A_{nb} = 0.78 \times \frac{\pi (d^2)}{4} = 245.04\,mm^2.$

$= \frac{1}{1.25}\left[\frac{400}{\sqrt{3}}\left[2 \times 245.04\right]\right] \times 0.9$

$V_{nsb} = 81.47\ KN$

ii) Design Bearing Strength

$V_{dpb} = \frac{1}{\gamma_{mb}}\left[2.5 \times k_b \times d \times t \times f_u\right]$ $\quad e = 1.7\,d_0 = 37.4 \approx 40mm$

$P = 2.5\,d = 50mm$

$k_p = \frac{P}{3d_0} - 0.25 = 0.50 \quad \therefore k_b = \frac{e}{3d_0} = 0.6$

$\therefore k_p = 1 \qquad k_b = \frac{f_{ub}}{f_u} = 1.025$

$= \frac{1}{1.25}\left[2.5 \times 0.5 \times 20 \times 18 \times 410\right]$

$V_{dpb} = 82\ KN$

$\therefore$ Bolt value $= 81.47\ KN$

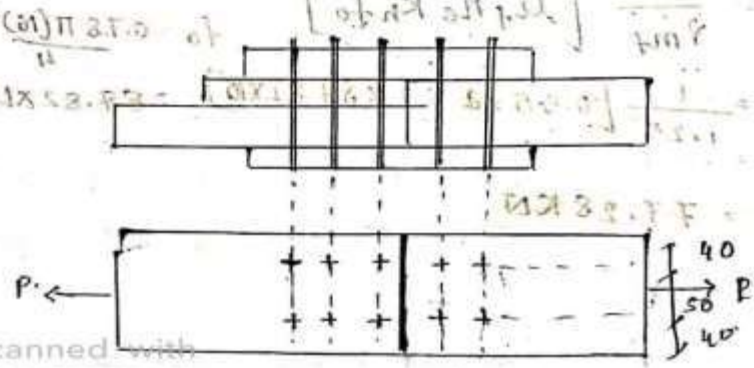
iii) No of bolts

$N = \text{No.} \frac{\text{Factored load}}{\text{Bolt value}} = \frac{750}{81.47} = 9.2$

Say 10 bolts : $\therefore$ provide 10 bolts

iv) Arrangement of bolts

$P = 50mm, \quad e = 40mm, \quad \text{Load} = 750\ KN$



Check for strength due to rupture of critical section.

$$T_{dn} = \frac{0.9 \times A_n \times f_u}{\gamma_m}, \qquad A_n = [b - n d_h] t$$

$$= (130 - 2 \times 22) \times 10$$

$$= 860\ mm^2$$

$$= \frac{0.9 \times 860 \times 410}{1.25}$$

$$T_{dn} = 256.968\ KN$$

$\therefore$ $T_{dn} <$ Factored Load

Hence Unsafe.

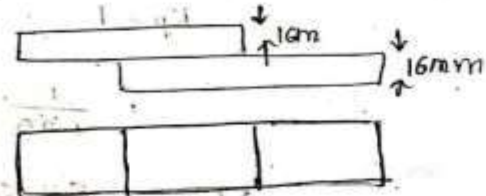
8] 2-plates 16mm thickness have been connected in lap joint using HSFG bolts design the joint so as to transmitt a pull equal to the full shear of the plate. Adopt 16mm dia bolts assume edge distance of 30mm & pitch = 60mm

Sol^n: $t = 16mm$

$d = 16mm \therefore d_o = 18mm$

$e = 30mm$

$P = 60\ mm.$



Assume property Class 8.8 $\therefore f_{ub} = 800\ N/mm^2$

$\gamma_{mb} = 1.25$

Assume Fe 410 plate, $f_u = 410 N/mm^2$

$f_y = 250\ N/mm^2$

Factored load = Pull strength.

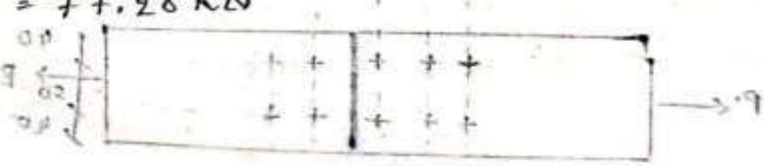
i] Design shear strength of HSFG bolts [Bolt value] (V_{dsb})

13.684

$$V_{dsf} = \frac{1}{\gamma_{mf}} \left[\mu_f n_e K_h f_o \right] \qquad f_o = 0.78 \frac{\pi (16)^2}{4} \times 0.70 \times f_{ub}$$

$$= \frac{1}{1.25} \left[0.55 \times 2 \times 1 \times 87.82 \times 10^3 \right] \qquad = 87.82 \times 10^3$$

$$V_{dsf} = 77.28\ KN$$



ii] Design Yield Strength of the plate

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} =$$

[iii] Design Rupture Strength of the plate

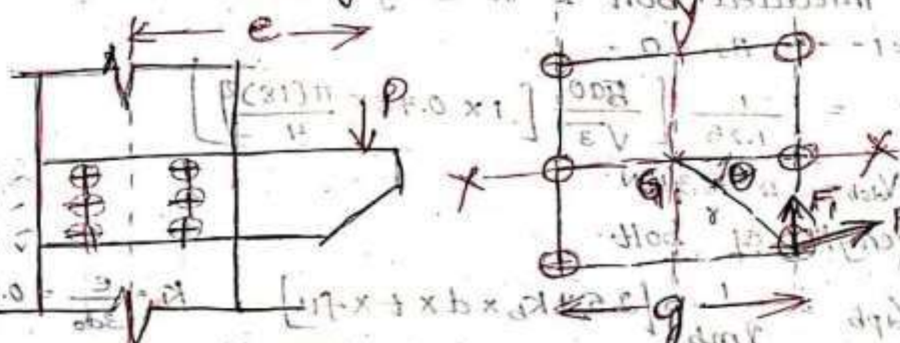
$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} \Big] x$$

Full strength = least of the above two.

iii] No of bolts = $\frac{\text{Factored load}}{\text{Bolt Value } (V_{dsb})}$

Bracket Connection [Bolted & eccentric action]

Type 1 : Load acting parallel to bolt group



Resultant force of F_1 & F_2 = $F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$

where F_1 = force due to axial load on each bolt.

i.e., $\boxed{F_1 = \frac{P}{N}}$ P = axial load, N = No of bolts

F_2 = Force due to bending / Torsion. $\boxed{F_2 = \frac{Mr}{\Sigma r^2}}$

where, $M = P \times e$ [Force × Eccentricity = Moment]

If $F_R >$ Bolt value [Unsafe]

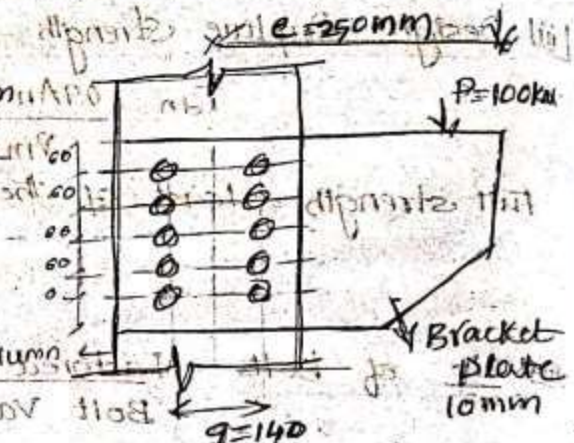
if $F_R <$ Bolt value [Safe]

Problems

1) Check safety of the bracket connection shown in fig. Use M18 & property class 5.6 bolts.

Sol^n: For M18, $d = 18mm$

$d_o = 18 + 2 = 20mm$



For P.C 5.6 → $f_{ub} = 500 N/mm^2$

$\gamma_{mb} = 1.25$

$P = 60$

$g = 140$

$e = 1.7 d_o$

$= 1.7 \times 20$

$e = 34$

say $e = 40$

Assume Fe 410 bracket plate, ∴ $f_u = 410 N/mm^2$

$f_y = 250 N/mm^2$

i) Find the shear strength of the bolt

$$V_{dsb} = \frac{1}{\gamma_{mb}}\left[\frac{f_{ub}}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]\right]$$

Assume fully threaded bolt & in single shear

∴ $n_n = 1$; $n_s = 0$

$$= \frac{1}{1.25}\left[\frac{500}{\sqrt{3}}\left[1 \times 0.78 \times \frac{\pi(18)^2}{4}\right]\right]$$

$$\boxed{V_{dsb} = 45.83 kN}$$

ii) Bearing strength of bolt

$$V_{dpb} = \frac{1}{\gamma_{mb}}\left[2.5 \times K_b \times d \times t \times f_u\right]$$

$K_b = \frac{e}{3d_o} = 0.67$

$K_b = \frac{P}{3d_o} - 0.25 = 0.75$

$K_b = \frac{f_{ub}}{f_u} = 1.025$

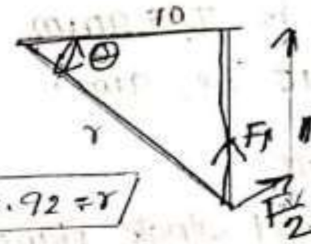
$$= \frac{1}{1.25}\left[2.5 \times 0.67 \times 18 \times 10 \times 410\right]$$

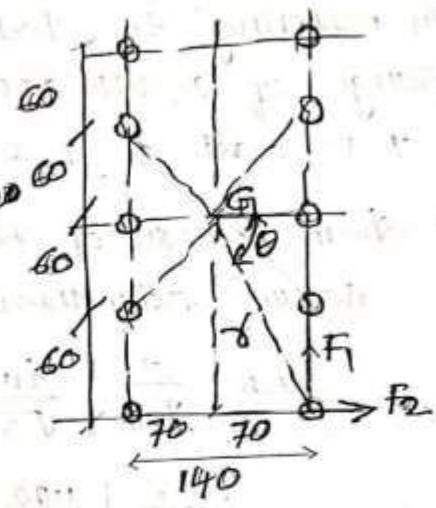
$$\boxed{V_{dpb} = 98.89 kN}$$

$$\boxed{\therefore \text{Bolt value} = 45.83 kN}$$

iii) Resultant force of Bolt

$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$





$\tan\theta = \dfrac{\text{adp}}{\text{adj}} = \dfrac{120}{70}$

$\boxed{\theta = 59.74}$

$r = \sqrt{70^2 + 120^2} = \boxed{138.92 = r}$

Factored load = 1.5 × service load

$= 1.5 \times 100$

$\boxed{P = 150\ KN}$

$\therefore M = P \times e = 150 \times 10^3 \times 250$

$\boxed{M = 37.5 \times 10^6\ N\text{-}mm}$

$\Sigma r^2 = 4(70^2 + 120^2) + 4(70^2 + 60^2) + 2(70^2 + 0)$

$\boxed{\Sigma r^2 = 121 \times 10^3}$

$F_1 = \dfrac{P}{N} = \dfrac{150 \times 10^3}{10}$ $\boxed{F_1 = 15 \times 10^3\ N}$

Torsional force. $F_2 = \dfrac{Mr}{\Sigma r^2} = \dfrac{(37.5 \times 10^6)(138.92)}{121 \times 10^3}$

$\boxed{F_2 = 43.05 \times 10^3\ N}$

$$F_R = \sqrt{(15 \times 10^3)^2 + (43.05 \times 10^3)^2 + 2(15 \times 10^3 \times 43.05 \times 10^3)\cos(59.74)}$$

$\boxed{F_R = 52.24\ KN}$

Compare Bolt values & F_R.

$F_R > BV$ [∴ The connection is unsafe]

---

2) A bracket plate bolted to a vertical column & is as shown in figure. If M-20 bolts of grade 4.6 are used determine the max value of factored load P which can be carried safely.

Sol^n: For M-20, bolts, $d = 20\,mm$

$d_0 = 20 + 2 = 22\,mm$

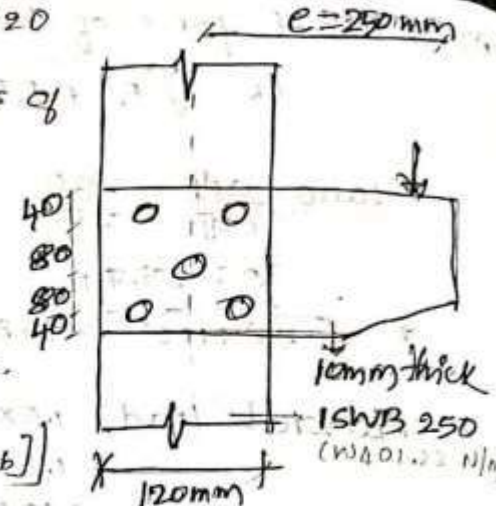
For P.C 4.6, $f_{ub} = 400\,N/mm^2$

$\gamma_{mb} = 1.25$

Pitch = 80 , end distance = 40 , $g = 120$

By reffering to steel table, thickness of flange of I.S.W.B 250 is $T_f = 9mm$

T is least of 9 & 10 i.e, 9mm



$n_n = 1, \ n_s = 0$

i) Shear strength of bolt

Assume fully threaded, single plane.

$$V_{dsb} = \frac{1}{\gamma_{mb}}\left[\frac{f_{ub}}{\sqrt{3}}\left[n_n A_{nb} + n_s A_{sb}\right]\right]$$

$$= \frac{1}{1.25}\left[\frac{400}{\sqrt{3}}\left[1 \times 0.78 \times \frac{\pi(20)^2}{4}\right]\right]$$

$$\boxed{V_{dsb} = 45.27 \text{ KN}}$$

ii) Bearing strength of bolt

$$V_{dpb} = \frac{1}{\gamma_{mb}}\left[2.5 \times K_b \times d \times t \times f_u\right]$$

$$= \frac{1}{1.25}\left[2.5 \times 0.60 \times 20 \times 9 \times 410\right]$$

$$\boxed{V_{dpb} = 88.56 \text{ KN}}$$

$K_b = \frac{e}{3d_0} = 0.60$

$K_b = \frac{P}{3d_0} - 0.25 = 1.21$

$K_b = \frac{f_u}{f_{ub}} = 1.025$

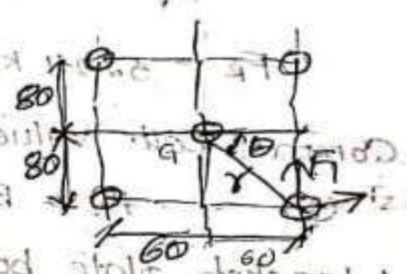
$K_b = 1.$

$\therefore$ Bolt value [least of the above 2] = 45.27 KN

iii) Resultant Force

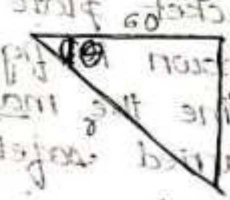
$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$

$r = \sqrt{(60)^2 + (60^2}$ $\quad r = 100 mm$



$\Sigma r^2 = 4(60^2 + 80^2) + 1(0+0)$

$\Sigma r^2 = 40 \times 10^3$



$M = P \times e = P \times 250$

$M = 250 P$

$\tan\theta = \frac{80}{60}$

$\theta = 53.13$

$F_1 = \frac{P}{N} = \frac{P}{5} = 0.2P$

$F_2 = \frac{Mr}{\Sigma r^2} = \frac{250 P \times 100}{40 \times 10^3} = 0.625 P$

$$F_R = \sqrt{(0.2P)^2 + (0.625P)^2 + 2(0.2P \times 0.625P)\cos(53.13)}$$

$$= \sqrt{0.04P^2 + 0.39P^2 + 0.15P^2}$$

$$F_R = 0.76P$$

$$F_R = BV$$

$$0.76P = 45.27$$

$$\boxed{P = 59.56\ KN}$$

DESIGN OF STEEL STRUCTURAL ELEMENTS (18CV61)

MODULE 2
DESIGN OF WELDED CONNECTIONS

MODULE 02 - WELDED CONNECTIONS

Welding is a method of connecting between two pieces of metal by heating to a plastic or fluid state (with or without pressure), so that fusion occurs.

Welding is the most efficient and direct way of connecting the metal pieces. Over many decades, different welding techniques have been developed to join metals.

Welding is generally performed by either electric or gas. Most of the welding is done using electric supply. Through gas welding is relatively cheaper, it is a slow process. Hence this method is generally used for repair and maintenance purpose.

Welding Process :

- ✓ In the most common processes of welding structural steel, electric energy is used as the heat source.
- ✓ Electric welding involves passing either direct or alternate current through on electrode.
- ✓ By holding the electrode at a very short distance from the base metal which is connected to one side of the circuit, an arc forms as the circuit is essentially shorted.
- ✓ With this shorting of the circuit, a very large current flow takes place which melts the electrodes tip (at the arc) and the base metal .

- ✓ A temperature of 3300- 5000 degree Celsius is produced in the arc.
- ✓ The electrons flow making the circuit carries the molten electrode metal to the base metal to build up the joint.
- ✓ The parameters that control the quality of weld are the electrode size and the current that produces sufficient heat to melt the base metal.
- ✓ The different processes of arc welding that are used in structural steel applications are as follows
  - i. Shielded metal arc welding
  - ii. Submerged arc welding
  - iii. Gas shielded metal arc welding
  - iv. Flux core arc welding
  - v. Electro slag welding
  - vi. Stud welding

ADVANTAGES OF WELDING:

1. Drilling of holes are eliminated
2. Welding joints are air tight and water tight
3. Welded connection gives good aesthetic appearance
4. It is possible to achieve 100% efficiency in the joint where as in bolted connection it reaches maximum of 70 – 80%
5. Noise produced in welded process is relatively less
6. Any shape can be connected
7. Tubular sections can be connected
8. A truly continuous structure can be made
9. Alterations in connections can be made in the design of welded connections.

ADVANTAGES OF WELDING

1. Welding requires skilled labours
2. Costly equipment is required
3. Welded joints are over rigid
4. Proper welding in field condition is difficult
5. Inspection of welded connection is difficult
6. Continuous power supply is required
7. Brittle failure is more in welded connections

TYPES OF WELDED JOINTS:

There are three types of welded joints

1) Butt Weld

It is also known as groove weld. Depending upon the shape of groove made for welding butt welds are classified as follows

- Square butt weld on one side
- Square butt weld on both side
- Single butt weld
- Double V-butt weld
- Single J-butt weld
- Single dowel butt weld

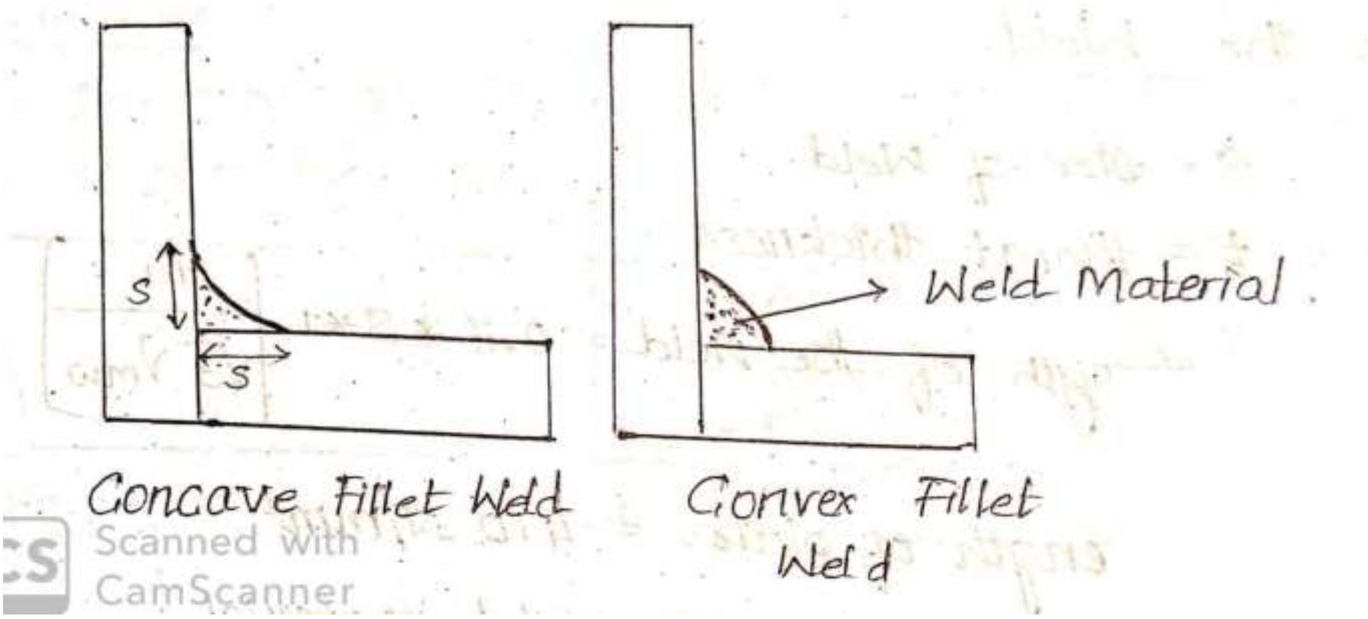
Single V-butt weld

Double V-butt weld

2) Fillet Weld:

- It is a weld of approximately triangular cross section joining two surfaces at right angles to each other in lap joint or T-joint or corner joint as shown in figure.
- When the cross section of the fillet weld is isosceles triangle , it is known as standard fillet weld,. In special circumstances 60 degree or 30 degree are also used.
- A fillet weld is also known as concave fillet weld/convex fillet weld depending upon the shape of the weld shape as shown in figure

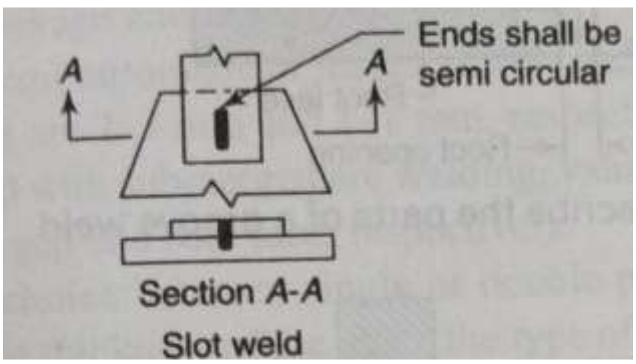
Typical Fillet Weld

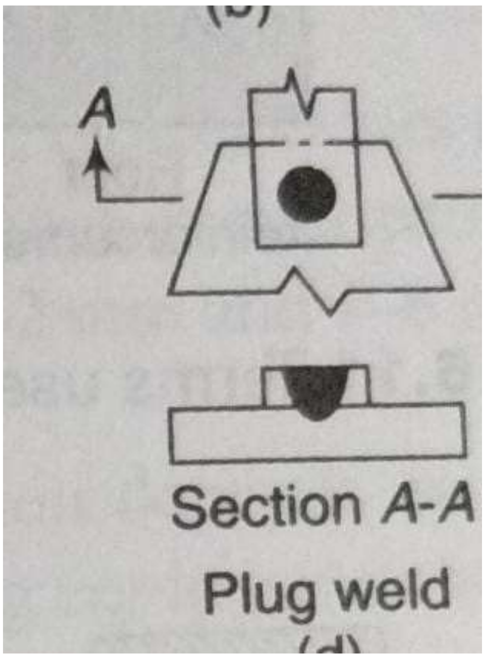


3) Slot Weld and Plug Weld

In slot type of weld a plate with circular hole Is kept on another plate to be joined and then fillet welding is made along the circumference of the hole.

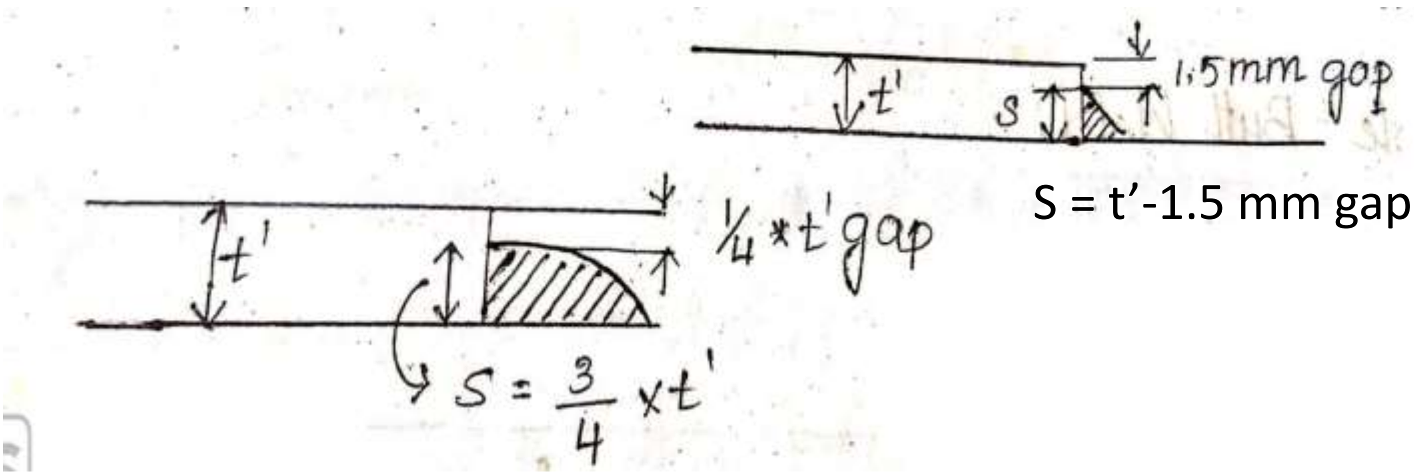
In plug weld small holes are made in one plate and kept over another plate to be connected and the entire hole is filled with filler material





WELD SPECIFICATIONS [Page No.: 78 IS 800 – 2007]

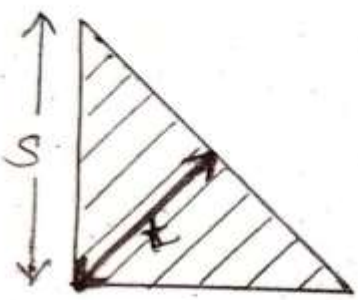
1. Minimum size of the Weld = 3mm
2. Maximum size of the Weld = S = (t' – 1.5mm gap)



3. Length of weld should be greater than width of plate.
4. End return length of weld = 25mm
5. For intermittent length = 45 or 40 mm
6. Lap length = 4 * plate thickness or 40 mm
7. Strength of the fillet weld

$$= 0.7 \times S \times L \times [fu/\sqrt{3}\gamma_{mw}]$$

where fu = ultimate strength of plate = 410 N/mm^2



γ_{mw} = Partial safety factor for weld material. [page 30]

= 1.25 for shop fabrication

= 1.5 for field fabrication

S = Size of weld

t' = Thickness of thinner plate.

8. Strength of butt weld

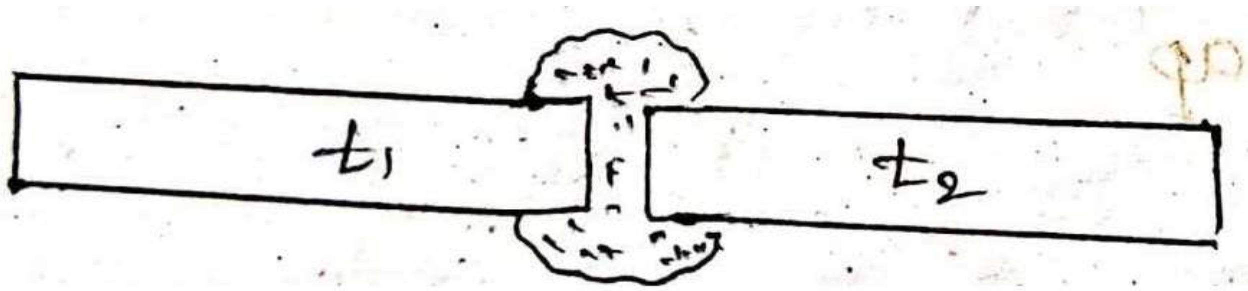
a) Single Butt Weld

Strength of Single Butt weld

$$= 5/7 \times t' \times L \times [fu/\sqrt{3}\gamma_{mw}]$$

where t' = Thickness of thinner plate.

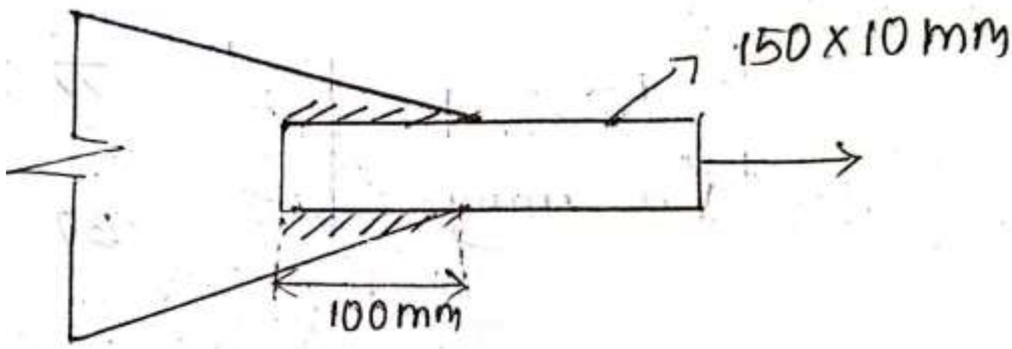
b) Double Butt Weld



Strength of double Butt weld = $t' \times L \times [fu/\sqrt{3}\gamma_{mw}]$

PROBLEMS :

1. Determine strength of the welded joint for the diagram shown in figure.



Sol: Strength of fillet Weld $= 0.7 \times S \times L \left[\frac{f_u}{\sqrt{3}\,\gamma_{mw}}\right]$

Length of Weld $= 100 + 100 = 200\,mm$

Assume Fe 410 plate. **and** shop welding

$\therefore f_{ub} = 410\,N/mm^2$

$\gamma_{mw} = 1.25$

Size of Weld, $S = t' - 1.5$ gap $= 10 - 1.5$

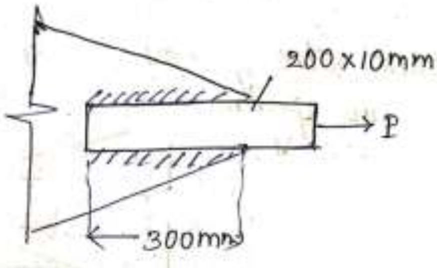
$S = 8.5\,mm$ say $8\,mm$

Strength of fillet Weld $= 0.7 \times 8 \times 200 \left[\frac{410}{\sqrt{3} \times 1.25}\right]$

$= 212\,KN$

2. Two plates are connected by a fillet weld using 8mm welding size. Welding is provided on two sides with a length of 300 mm as shown in figure. Find the strength of the weld.

If weld is provided on three sides what is percentage increase in the strength of the weld.



Soln: t = 8mm,L = 300+300= 600mm for 2 side welding

L = 300 + 300 + 200 = 800 mm for 3 side welding.

Assume Fe 400 plate and shop welding

Therefore fu = 400 N/mm2 and ϒmw = 1.25

Strength of Weld for side $= 0.7 \times S \times L \left[\frac{f_u}{\sqrt{3}\ \gamma_{mw}}\right]$

$= 0.7 \times 8 \times 600 \times \left[\frac{410}{\sqrt{3} \times 1.25}\right]$

$= 636.28$ KN

For 3-side welding, length of welding, $= 800$ mm

Strength for 3 side welding $= 0.7 \times S \times L \times \left[\frac{f_{ub}}{\sqrt{3}\ \gamma_{mw}}\right]$

$= 0.7 \times 8 \times 800 \times \left[\frac{410}{\sqrt{3} \times 1.25}\right]$

$= 848.38$ KN

∴ % increase in strength w.r.t 3 side welding

$= \frac{848.38 - 636.28}{848.38} \times 100$

$= 25\%$

3. 18 mm thick plate is joined to a 16mm thick plate and 200mm long butt weld. Determine the strength of the joint if a

   a. Single V Butt weld is used.

   b. A double V butt weld is used.

Soln: Assuming Fe 410 grade plate & shop fabrication

$f_u = 410\ N/mm^2$, $\gamma_{mw} = 1.25$

Length of weld, $L = 200mm$.

a) for a single V-butt Weld.

$$\text{Strength} = \frac{5}{8} \times t' \times L \left[\frac{f_u}{\sqrt{3}\ \gamma_{mw}}\right]$$

$t' = 16mm$

$$= \frac{5}{8} \times 16 \times 200 \times \left[\frac{410}{\sqrt{3} \times 1.25}\right]$$

$$= 378.74\ kN$$

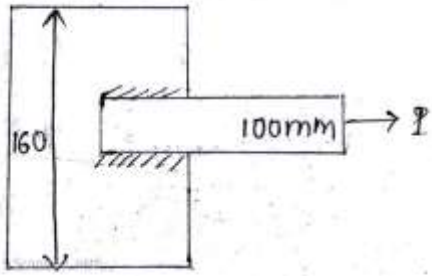
b) Double V-butt Weld

$$\text{Strength} = t' \times L \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$= 16 \times 200 \times \frac{410}{\sqrt{3} \times 1.25}$$

$$= 605.9\ kN$$

4. Design a suitable longitudinal fillet weld to connect the plates as shown in figure to transmit the pull equal to full strength of small plate. Plates are 12 mm thick and the grade of plate is Fe400 and welding to be made in workshop.



Sol: Length of Weld, $L = ?$

Assuming Fe 4 for Fe 410 plate

$f_u = 410 N/mm^2$

$\gamma_{mw} = 1.25$

$t = 12mm$

Size of Weld, $S = t - 1.5$ gap $= 12 - 1.5 = 10.5$

say 10 mm

Full strength of smaller plate, $T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$

$A_g = b \times d = 100 \times 12 = 1200 mm^2$

$\frac{1200 \times 250}{1.10}$

$T_{dg} = 272.2$

Design strength $T_{dw} = 0.7 \times S \times L \left(\frac{f_u}{\sqrt{3}\, \gamma_{mw}}\right)$

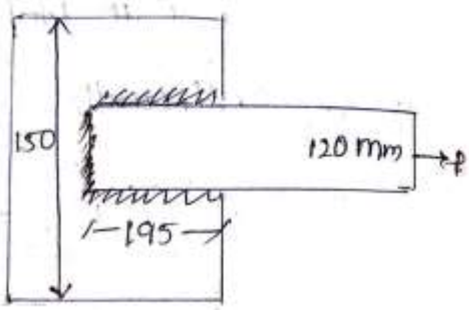
$272.2 \times 10^3 = 0.7 \times 10 \times L \times \frac{410}{\sqrt{3} \times 1.25}$

$L = 205.7 mm$

Say length of Weld $= 210$ mm.

Length of each weld on each side $= \frac{210}{2} = 105mm$

5. Determine the size of weld, pull transmitted, length of the weld and tensile strength of plate of smaller plate, for plates shown in figure, if plates are 10 mm thick each. Assume suitable partial safety factor and yield stress for welded steel plate.



Soln: $S = ?$

$P = ?$

length of Weld, $L = ?$

$T_{dg} = ?$

Assuming Fe 410 plates & shop weld

$f_u = 410 \ N/mm^2$, $\gamma_{mw} = 1.25$. $f_y = 250 \ N/mm^2$

$t = 10mm$

a) length of Weld, $L = 195 + 195 + 120$

$L = 510 \ mm$

b) Pull Transmitted / Tensile strength of plate (smaller plate)

$$T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}} = \frac{120 \times 10 \times 250}{1.10}$$

$$T_{dg} = 272.72 \ kN$$

∴ Maximum pull exerted = $T_{dg} = 272.72 \ kN$

c) Size of the Weld,

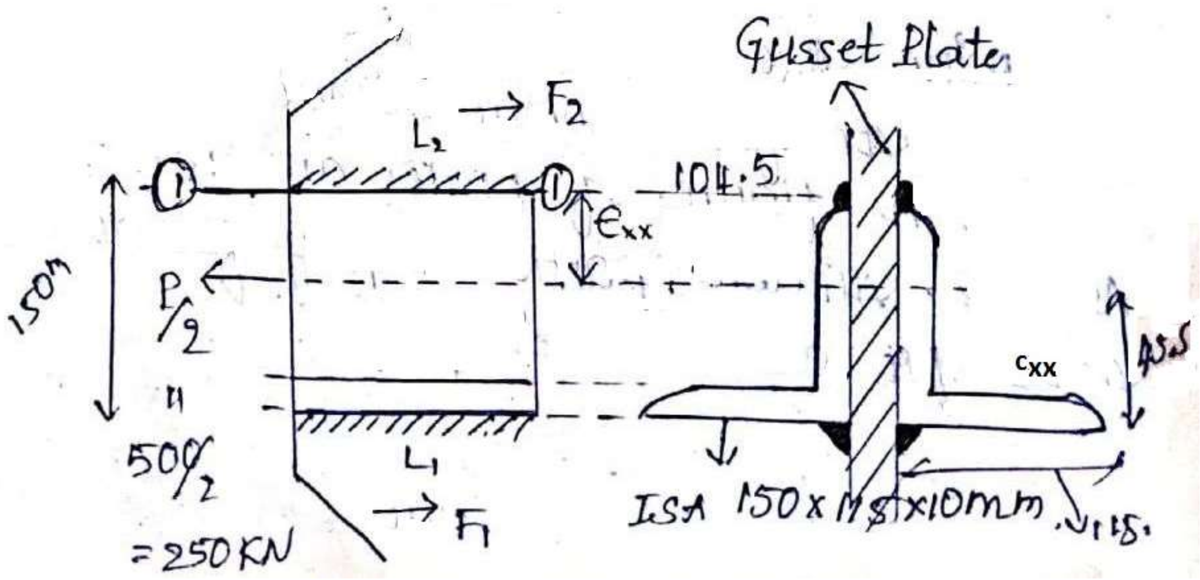
Equating T_{dg} to Design strength

$$272.72 \times 10^3 = 0.7 \times S \times 510 \left[\frac{410}{\sqrt{3} \times 1.25}\right]$$

$S = 4.08 \ mm$ say $5mm$.

∴ provide size of weld = 5mm.

6. A tie member of roof roof consists of 2 ISA 150 x 150 x 10 mm angles. They are connected on either side to 10 mm gusset plate and the member is subjected to a factored tensile force of 500 KN. Design the welded Connection assuming the connection are made in workshop.



Factored load = 500KN

γ_{mw} = 1.25

Gusset plate thickness = 10mm. t = 10mm

Size of the weld = S = $\frac{3}{4}$ of angle thickness

$= 3/4 \times 10$

$S = 7.5$mm say 7mm

From steel tables $C_{xx} = 45.5$m, $E_{xx} = 104.5$

Since each angle is independ ∴ consider force =

$$\text{Force} = \frac{P}{2} = \frac{500}{2} = 250\text{KN}$$

a) Equating Force = Strength of the Weld.

$$250\times10^3 = 0.7\times7\times L\times\frac{f_u}{\sqrt{3}\times\gamma_{mw}}$$

$$250\times10^3 = 0.7\times7\times L\times\left[\frac{410}{\sqrt{3}\times1.25}\right]$$

$$L = 269.49 \text{ say } L = 270\text{ mm}$$

$\therefore\ L = L_1 + L_2 = 270\text{ mm}$

b) Taking Moment about 1-1

$$F_1\times150 - \cancel{250}^{104.5}\times45.5 + \cancel{F_2\times0}^{\to0} = 0$$

$$F_1 = 174.167$$

$$0.7\times S\times l_1\times\frac{f_y}{\sqrt{3}\,\gamma_{mw}}\times150 - 250 - 45.5 = 0$$

$$0.7\times7\cdot\times l_1\times\frac{410}{\sqrt{3}\times1.25}\times150 - 250 - 45.5 = 0$$

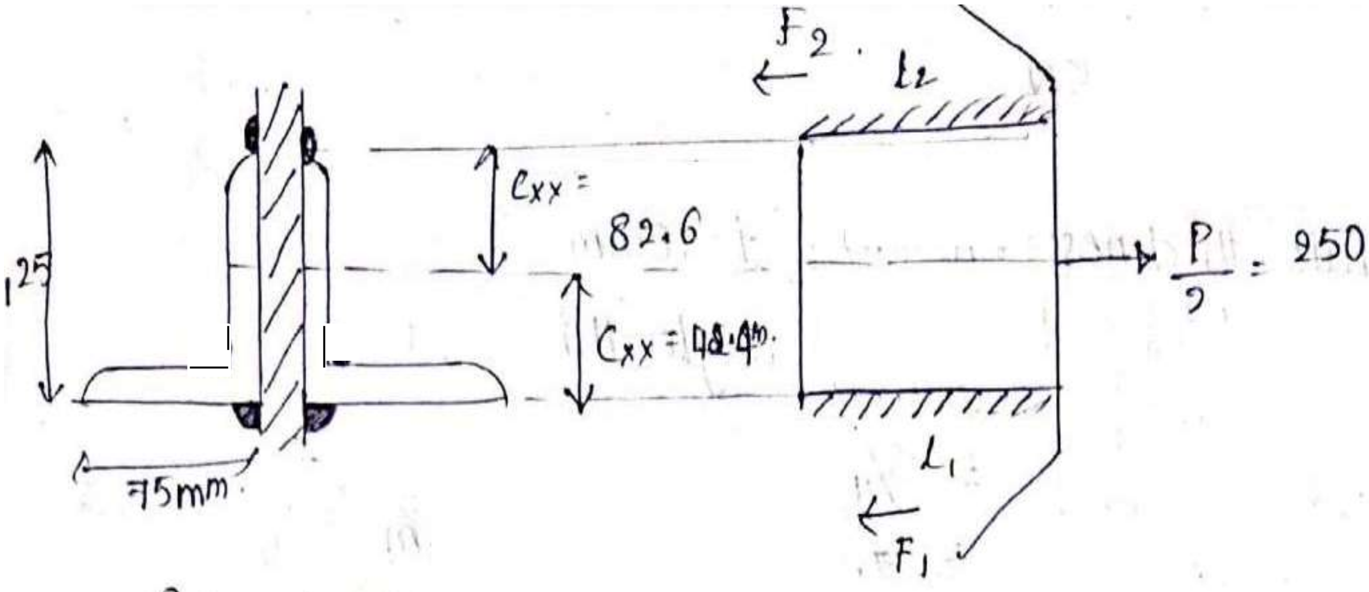
$$l_1 = 81.72 \simeq 80\text{mm}$$

$\therefore\ l_1 + l_2 = 270$

$$81.72 + l_2 = 270$$

$$l_2 = 188.28\text{ mm. Say } 190\text{ mm.}$$

7. A tie member of roof truss consist of 2 ISA 125 x 75 x 10 mm. the tie member is subjected to a pull of 250 KN. The angles are connected on either side of a gusset plate of 10 mm thick with long legs back to back. Design the end connection assuming g the fillet weld.



$\gamma_{mw} = 1.25$

Gusset plate thickness = 10mm, t = 10mm

Size of the weld, $S = 3/4 \times 10$

$S = 7.5$ mm say 7mm

From steel table, $C_{xx} = 42.4$, E_{xx} = 104.5mm 82.6mm

Service load = 250.

Factored load = $1.5 \times 250 = 375$ KN

Force $= \frac{P}{2} = \frac{375}{2} = 187.5$

a) Equating Force = Strength of Weld.

$$187.5 \times 10^3 = 0.7 \times 7 \times L \times \left[\frac{410}{\sqrt{3} \times 1.25}\right]$$

$$L = 202.065 \text{ mm} \quad \text{Say } 245 \text{ mm}$$

$$\therefore L_1 + L_2 = 245 \text{mm}$$

b) Taking Moment about 1-1

$$F_1 \times 125 - 187.5 \times 82.6 + F_2 \times 0 = 0$$

$$F_1 = \cancel{836 \text{ kN}} \; 123.9$$

$$0.7 \times S \times L_1 \times \frac{f_u}{\sqrt{3} \times \gamma_{mw}} - 187.5 \times 82.6 = 0$$

$$L_1 =$$

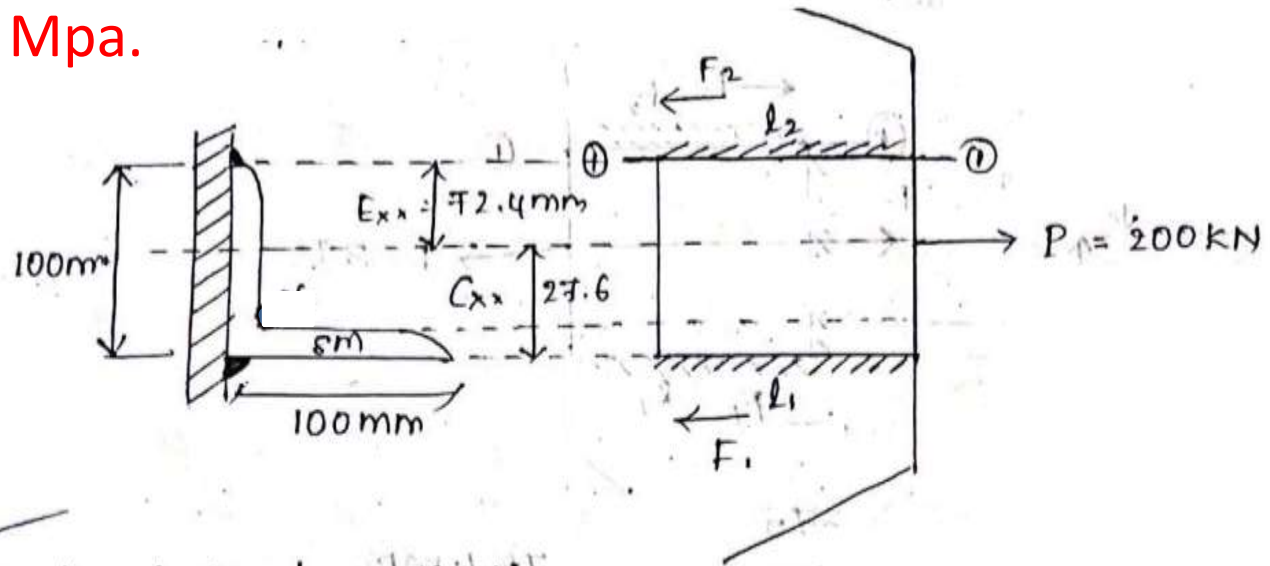
Say $L_1 = 160$ mm

$$L_1 + L_2 = 245$$

$$L_2 + 160 = 245$$

$$L_2 = 85 \text{mm}$$

8. In a truss angle 100 x 100 x 8mm is subjected to factored tension of 200 KN. It has to be connected to a gusset plate using fillet weld at the toe and back. Determine the weld length required so that centre of gravity of the welds lies in the plane of the centre of gravity of the angle. Take Fu 410 N/mm2 or Mpa.



Sol:

Factored load = 200 kN

Weld length = ?

$f_u = 410 N/mm^2$

$\gamma_{mb} = 1.25$ for shop fabrication

$t = 8mm$

Size of weld $= \frac{3}{4} \times 8 = 6mm.$

From steel table $C_{xx} = 27.6 mm$, $E_{xx} = 72.4 mm$

a) Equating Force = Strength of the weld

$$200 \times 10^3 = 0.7 \times 8 \times L \times \frac{410}{\sqrt{3} \times 1.25}$$

$L = 251.45 mm$ say 250 mm

$\therefore L_1 + L_2 = 250$

b) Taking Moment about 1-1

$$(F_1 \times 100) - (200 \times 72.4) + (F_2 \times 0) = 0$$

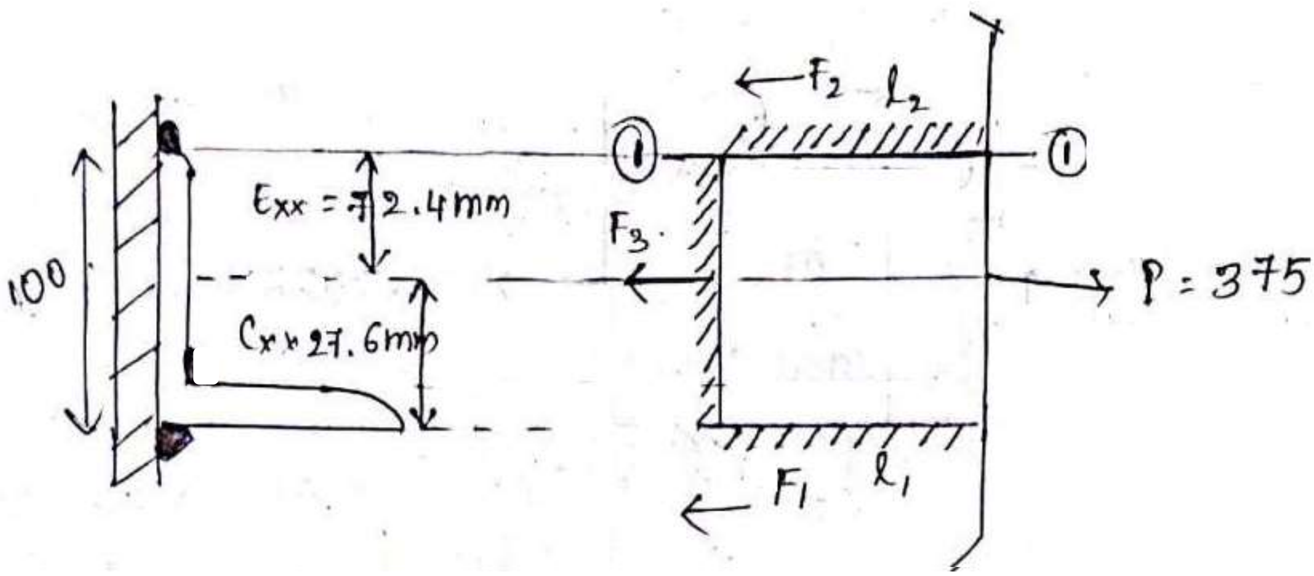
$$F_1 = 144.8 \text{ KN}$$

$$0.7 \times S \times L_1 \times \frac{f_u}{\sqrt{3}\,\gamma_{mw}} - 200 \times 7.244 = 0$$

$$L_1 = 182 \text{ mm.}$$

$$L_2 = 68 \text{ mm}$$

9. Design a Welded connection for an angle ISA 100 x 100 x 8mm subjected to a load of 250 KN. Provide 3 sides welding. Angle section has to be connected to a gusset plate using fillet weld. Assume suitably any missed data.



Factored load = $1.5 \times 250 = 375$

From steel table. $C_{xx} = 27.6\,mm$, $E_{xx} = 72.4\,mm$

Assume Fe 410 plate. $f_u = 410\,N/mm^2$, $\gamma_{mw} = 1.25$

$S = \frac{3}{4} \times 8 = 6\,mm$

a) Equating Force = Strength of Weld.

$$375 \times 10^3 = 0.7 \times 6 \times L \times \frac{410}{1.25 \times \sqrt{3}}$$

$L = 477.48\,mm$ say

$L = 480\,mm$.

$\therefore L_1 + L_2 + L_3 = 480\,mm$

$L_1 + L_2 = 480 - 100$

$L_1 + L_2 = 380\,mm \longrightarrow (i)$

b) Taking moment about (1)-(1)

$$F_1 \times 100 - 375 \times 72.4 - F_3 \times 72.4 + F_2 \times 0 = 0$$

$$F_1 \times 100 - 27150 \times 10^3 - 72.4 \times F_3 = 0$$

$$\left[0.7 \times 6 \times L_1 \times \frac{410}{\sqrt{3} \times 1.25}\right] \times 100 - 27150 - \left[0.7 \times 6 \times 100 \times \frac{410}{\sqrt{3} \times 1.25}\right] = 0$$

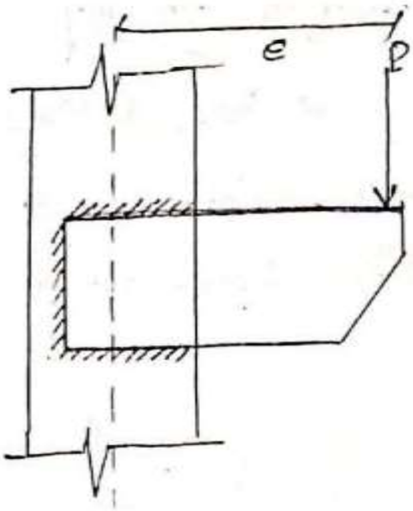
$L_1 = 268\,mm \simeq 270\,mm$

$L_1 + L_2 = 380\,mm$

$L_2 = 110\,mm$

BRACKET WELDED CONNECTION

Type 1 – Bracket load acts parallel to weld group:



Resultant force $\not>$ Permissible stress in the weld

$$F_R \not> \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$\sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta} \not> \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$F_1 = \frac{P}{l \times t} \quad ; \quad F_2 = \frac{M \times r}{I_p} \quad ; \quad I_p = I_{xx} + I_{yy}$$

$$I_{xx} = \frac{bd^3}{12} + Ah_i^2$$

$$I_{yy} = \frac{\cancel{bd^3}\,db^3}{12} + Ah_i^2$$

where P = Factored load

L = Length of the weld

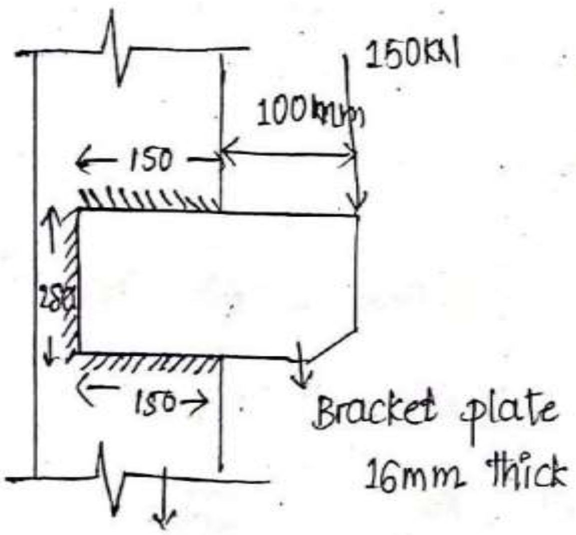
M = Moment = $P \times e$

I_p = Polar moment of inertia

F_1 = Force due to load

F_2 = force due to torsion

1. A bracket plate of thickness 16mm is welded to the flange of a column ISHB 400 at 759 N/M to support a load of 150 KN as shown in figure. Determine the size of the weld that could be required to support a load.



ISHB 400 at 759 N/m

Given:

thickness of plate, t = 16mm

Service load P = 150 kN

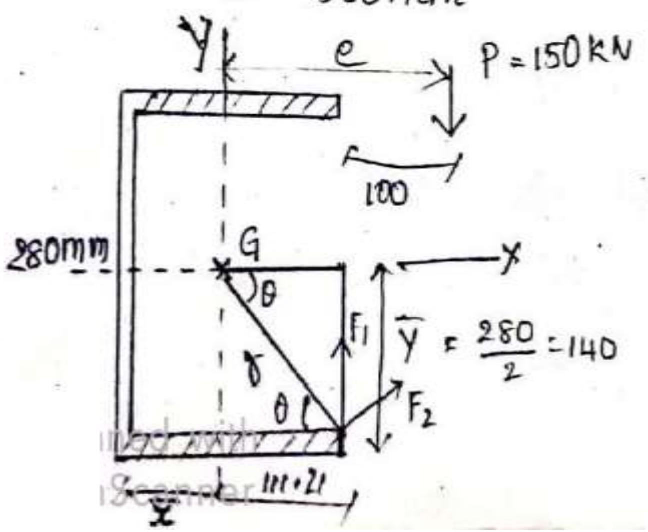
∴ Factored load = 1.5 × 150
= 225 kN

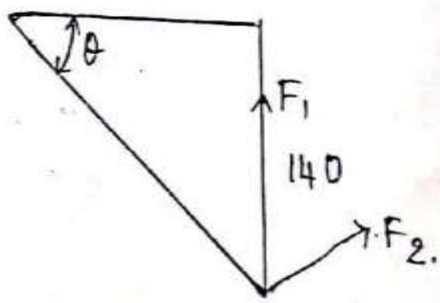
s = ?

t, throat thickness = 0.7 × s

Total length = 150 + 280 + 150

L = 580mm





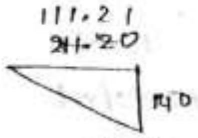
Location of Y-Y axis

$$\bar{x} = \frac{a_1 x_1 + a_2 x_2}{a_1 + a_2} = \frac{[2(150\times t)\times 75] + [250\times t]\times 0}{2[(150\times t)] + [(280\times t)]}$$

$$= \frac{2(150t)\times 75}{2(150t)+(280)} = 38.79$$

$\therefore$ eccentricity $= 100 + 111.21$

$= 211.20\,m$



$\tan\theta = \frac{140}{111.21}$

$\boxed{\theta = 51.54°}$

$$r = \sqrt{(111.20)^2 + (140)^2}$$

$r = 178.80\,m$

$\therefore$ Moment : $M = P\times e = 225\times10^3 \times 211.20$

$M = 47.52\times10^6$ N-m

$$I_{xx} = \frac{bd^3}{12} + Ah_1^2$$

$$= 2\left[\left[\cancel{\frac{150\times t^3}{12}}\right]^{\to 0} + [150\times t]\times 140^2\right] + \left[\cancel{\left[\frac{280\times t^3}{12}\right]}^{\to 0} + [\cancel{280\times t}]\times 0\right]$$

NOTE Since t is very small when compared to total length, neglect $t/2$, t^2, t^3 to simplify calculation

$I_{xx} = 7.71\times10^6 t$

$$I_{yy} = 2\left[\frac{t\times 150}{12} + (t\times150) * (75-38.8)^2\right] + \left[\frac{280t^3}{12} + (t\times280) * 38.8^2\right]$$

$I_{yy} = 1.38\times10^6 t$

$I_p = (7.71\times10^6\times t)(1.38\times10^6\times t)$

$\boxed{I_p = 9.09\times10^6 t}$

Direct stress. $F_1 = \frac{P}{L\times t} = \frac{225\times10^3}{580\times t} = \frac{387.93}{t}$

$$F_2 = \frac{M\times r}{I_p} = \frac{47.52\times10^6\times178.81}{9.09\times10^6 t} = \frac{934.72}{t}$$

$$F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta} \ngtr \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

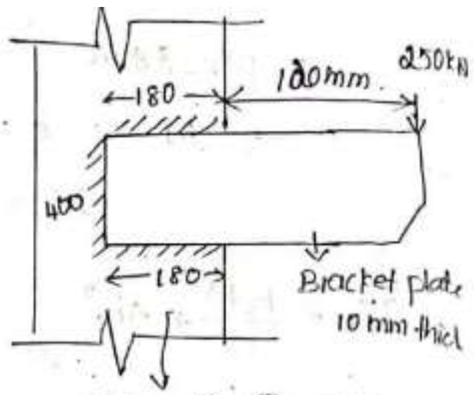
$$\sqrt{\left(\frac{387.93}{t^2}\right)^2 + \frac{(934.72)^2}{t^2} + 2\left[\frac{387.93\times934.72}{t^2}\right]\times\cos(51.54} \geq \frac{410}{1.25\sqrt{3}}$$

$t = 6.41$

$\therefore \; t = 0.7 \times S$

$S = \dfrac{t}{0.7} = \dfrac{6.41}{0.7} = 9.157$ say 10mm

2. The 10 mm thick bracket plae shown in figure is connected with the flange of the column ISHB 300@577 N/m. Find the size of the weld to transmit a factored load of 150 KN.



ISHB 300@ 577mm

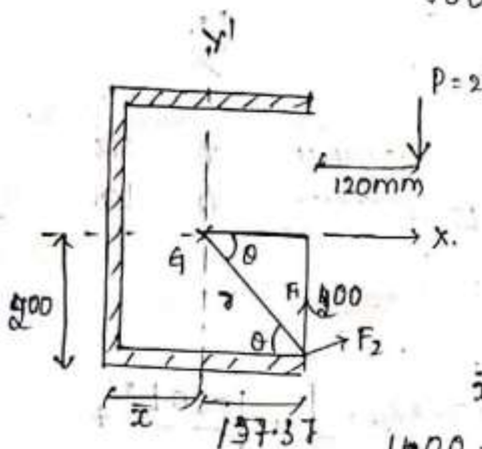
Sol^n. thickness of plate = 10mm

Factored load = 250 KN.

throat thickness = $0.7 \times S$

Total length = $180 + 400 + 180$

$= 760$mm



Location of Y-Y axis

$\bar{x} = \dfrac{a_1x_1 + a_2x_2}{a_1 + a_2} = \dfrac{2[(180\times t)\times 90] + [400\times t]}{2[(180\times t)+(400\times t)]}$

$\bar{x} = 42.63$

$180 - \bar{x} = 180 - 42.63 = 137.37$

$\therefore$ eccentricity $= 120 + 137.37 = 257.37$

$\tan\theta = \dfrac{200}{257.37} \Rightarrow \theta = 37.85$

$r = \sqrt{(200)^2 + (137.37)^2}$

$r = 242.63$ mm

Moment. $M = P \times e = 250 \times 257.37$

$$I_{xx} = \frac{bd^3}{12} + Ah_1^2$$

$$= 2\left[\left[\frac{180 \times t^3}{12}\right]^{20} + [180 \times t) \times 200^2\right] + \left[\frac{400^3 \times t}{12} + [400 \times t] \times 70\right]$$

$$= 14.4 \times 10^6 t + 5.33 \times 10^6 t$$

$$I_{xx} = 19.73 \times 10^6 t$$

$$I_{yy} = \frac{db^3}{12} + A_1 h_1^2 = 2\left[\left[\frac{t \times 180^3}{12}\right] + [t \times 180) \times [90 - 42.63]^2\right]$$

$$+ \frac{400t^3}{12} + (t \times 400)\, 42.63^2]$$

$$= 972 \times 10^3 t + 807.8 \times 10^3 + 726.92 \times 10^3$$

$$I_{yy} = 2.5 \times 10^6 t$$

$$I_p = I_{xx} + I_{yy} = 19.73 \times 10^6 t + 2.5 \times 10^6 t$$

$$I_p = 22.23 \times 10^6 t$$

$$F_1 = \frac{P}{Lt} = \frac{250 \times 10^3}{760 \times t} = \frac{328.94}{t}$$

$$F_2 = \frac{Mr}{I_p} = \frac{64.34 \times 10^6 \times 249.63}{22.23 \times 10^6 t} = \frac{702.24}{t}$$

$$\sqrt{\left[\frac{328.94}{t}\right]^2 + \left[\frac{702.24}{t}\right]^2 + 2\left[\frac{328.94}{t} \times \frac{702.24}{t}\right]\cos(37.85)} \ngtr$$

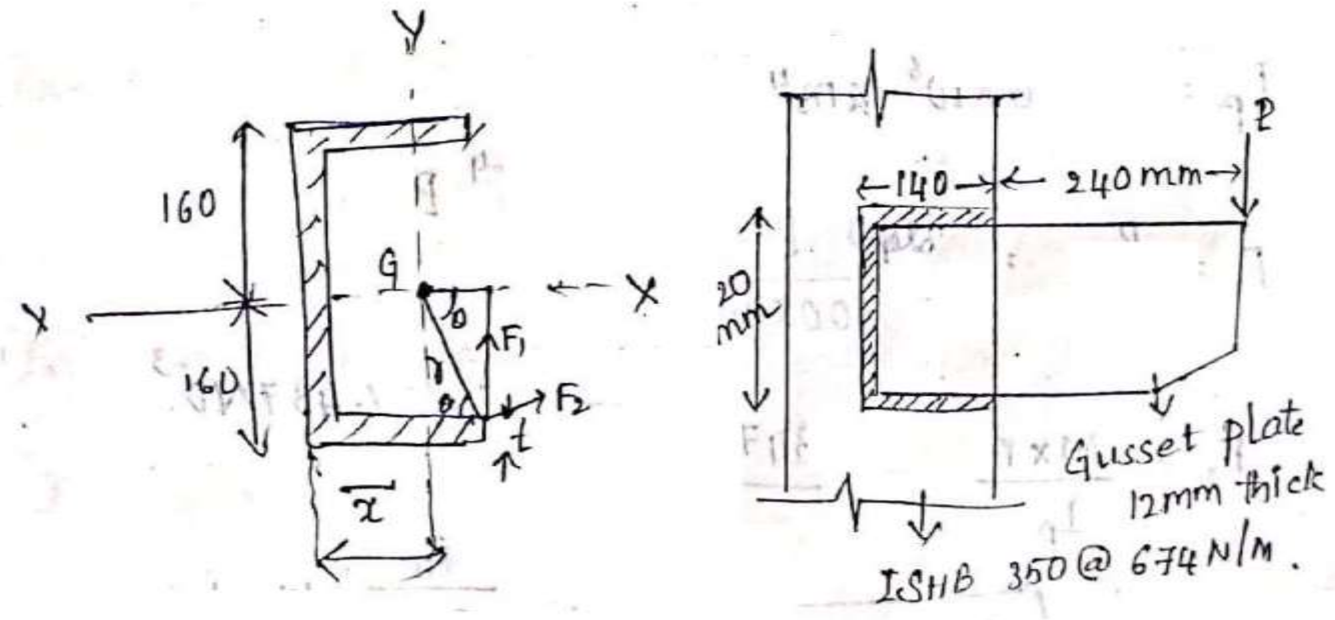
189.37

$$\frac{410}{\sqrt{3} \times 1.25}$$

$$\boxed{t = 5.19\,mm}$$

say $t = 5\,mm$ or $6m$

3. Calculate the load that can be transmitted through the eccentric welding connection shown in figure. Weld size = 6mm.



$$S = \frac{t}{0.7} \Rightarrow 6 = \frac{t}{0.7} \Rightarrow t = 4.2\,mm.$$

$$\bar{x} = \frac{a_1 x_1 + a_2 x_2}{a_1 + a_2}$$

$$= \frac{2(140 \times 4.2 \times 70) + (320 \times 4.2 \times 0)}{2(140 \times 4.2) + (320 \times 4.2)}$$

$$\bar{x} = 32.67\,mm$$

$$\text{Eccentricity} = 240 + 107.33 = 347.33\,mm$$

$$\tan\theta = \frac{160}{107.33} \qquad \theta = 56.14^\circ$$

$$r = \sqrt{(160)^2 + (107.33)^2} \qquad r = 192.66\,mm.$$

$$\text{Moment, } M = P \times e = P \times 347.33$$

$$I_{xx} = \frac{bd^3}{12} + Ah_1^2$$

$$= 2\left[\cancel{\frac{140 \times (4.2)^3}{12}}^{\,\to 0} + (140 \times 4.2) \times 160^2\right] + \left[\left[\frac{4.2 \times 320^3}{12}\right] + \left[\cancel{4.2 \times 320 \times 0}^{\,0}\right]\right]$$

$$I_{xx} = 41.57 \times 10^6\ mm^4$$

$I_{yy} = 4.99 \times 10^6 \, mm^4$

$I_p = I_{xx} + I_{yy}$

$= [41.57 + 4.99] \times 10^6$

$I_p = 46.56 \times 10^6 \, mm^4$

$F_1 = \frac{P}{L \times t} = \frac{P}{600 \times 4.2} = 3.96 \times 10^{-4} P$

$\frac{M \times r}{I_p} = \frac{347.33P \times 192.66}{46.56 \times 10^6} = 1.437 \times 10^{-3} P$

$F_R = \sqrt{F_1^2 + F_2^2 + 2 F_1 F_2 \cos\theta} \ngtr \frac{f_u}{\sqrt{3} \gamma_{mw}}$

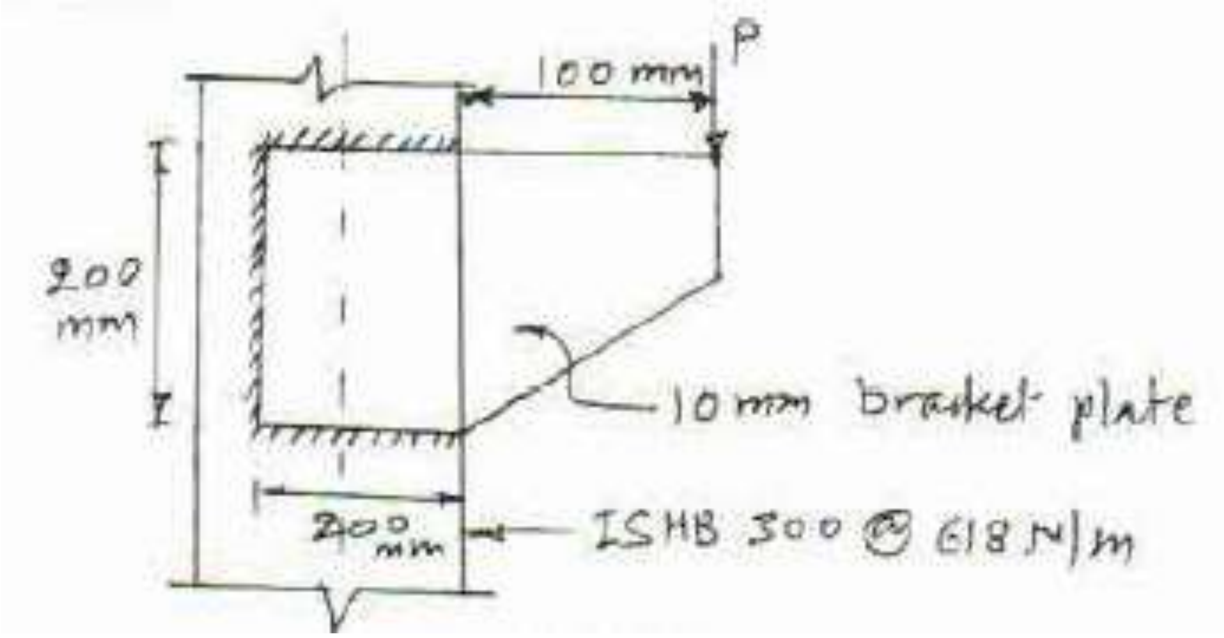
$\sqrt{(3.96 \times 10^{-4} P)^2 + (1.437 \times 10^{-3} P)^2 + 2(3.96 \times 10^{-4} P \times 1.437 \times 10^{-3} P) \cos(56.14)}$

$\ngtr \frac{410}{\sqrt{3} \times 1.25}$

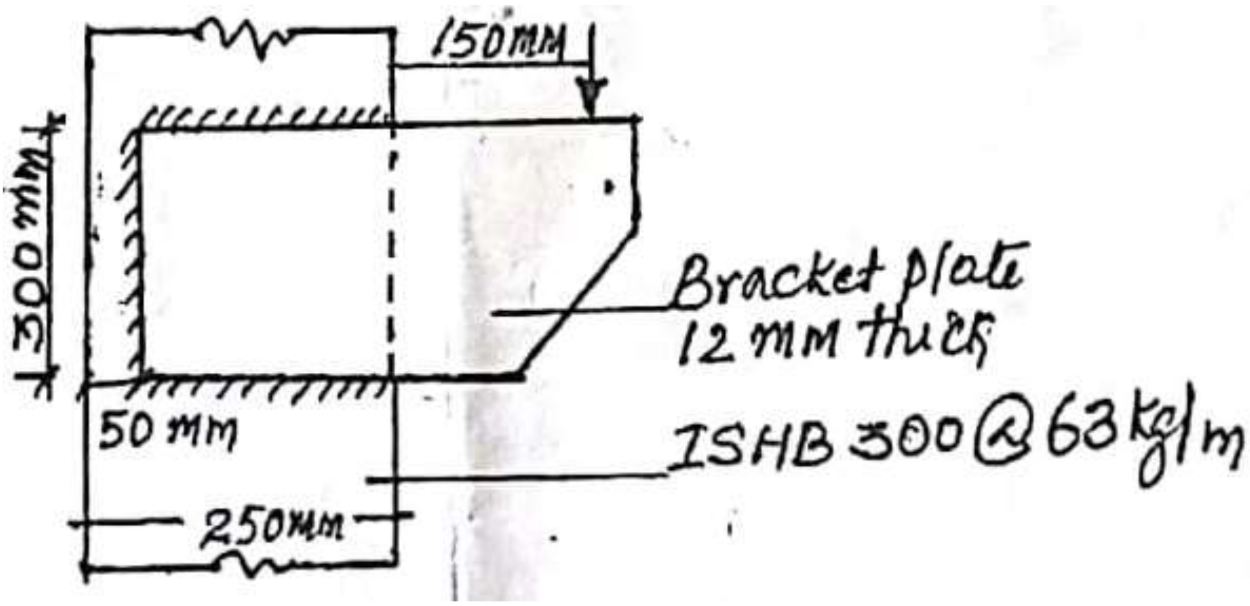
$1.683 \times 10^{-3} P = 189.4$

$P = 112.77 \, kN$

4. Determine the maximum load than can be registered by the bracket shown in figure by fillet weld of size 6mm.

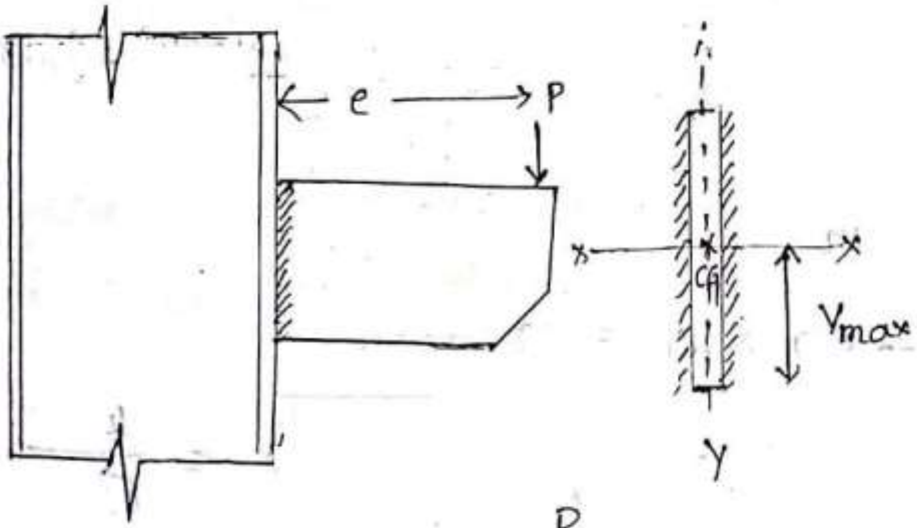


5. Calculate the factored load that can be supported by a bracket connection as shown in figure. Take size of weld as 6mm.



WELDED BRACKET CONNECTION

TYPE 2: Bracket load acting perpendicular to the Weld Group

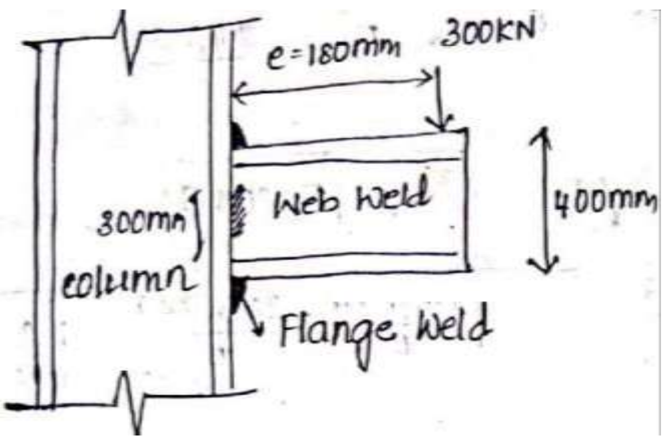


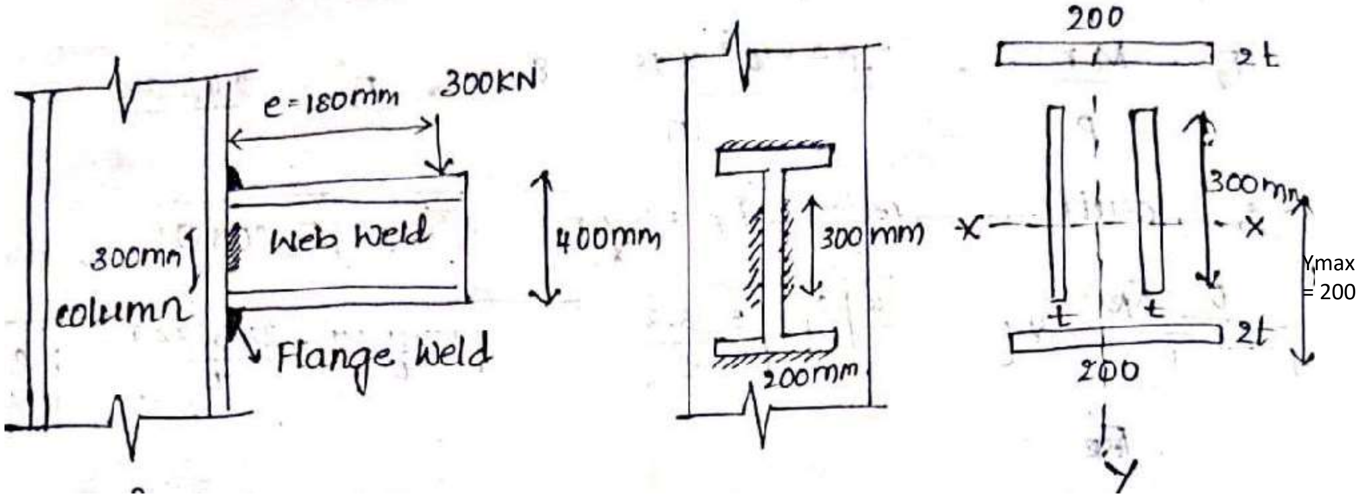
1) Direct stress, $F_1 = \frac{P}{L \times t}$

2) Bending stress, $F_2 = \frac{M}{I_{xx}} \times Y_{max}$

∴ Resultant stress $F_R = \sqrt{F_1^2 + F_2^2} \ngtr \frac{f_u}{\sqrt{3}\,\gamma_{wm}}$

1. A bracket of I section is welded to a steel stanchion by using flange weld as shown in figure as well as web weld as shown in figure. The size of the flange weld are double the size of the web. Determine the suitable weld size.





Sol^n: $S = ?$

$t = 0.6 \times S$

length of weld $= 2(200) + 2(300)$

$= 1000\,mm$.

Factored load $= 1.5 \times 300 \Rightarrow P = 450KN$

$e = 180mm$.

$$F_1 = \frac{P}{L \times t} = \frac{450 \times 10^3}{1000 \times t} = \frac{450}{t}$$

$$F_2 = \frac{M}{I_{xx}} \times Y_{max} = \frac{81 \times 10^6}{36.5 \times 10^6 t} \times 200 = \frac{443.83}{t}$$

$$\therefore I_{xx} = 2\left[\cancel{\frac{200 \times (2t)^3}{12}}^{0} + (200 \times 2t) \times 200^3\right] + 2\left[\frac{t \times (300)^3}{12} + (t \times 300) \times 0\right]$$

$I_{xx} = 36.5 \times 10^6 t\ mm^4$.

$M = P \times e = 450 \times 10^3 \times 180 = 81 \times 10^6$

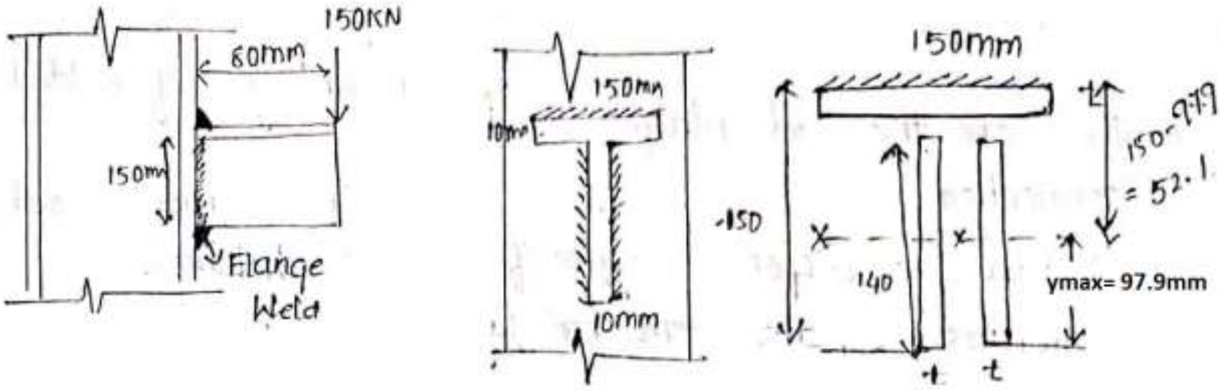
$\therefore$ Resultant Stress $F_R = \sqrt{\left[\frac{450}{t}\right]^2 + \left[\frac{443.83}{t}\right]^2} \geq \frac{410}{\sqrt{3} \times 1.25}$

$t = 3.34\,mm$

$t = 0.7 \times S$

$S = \frac{t}{0.7} = 4.77\,mm$

2. A bracket consisting of T section 150 x 150 x 10mm is connected to a column as shown in figure. The bracket carries 150 KN load at 80 mm eccentricity. Find out maximum throat thickness.



Sol^n: Location of x - x axis

$$= \frac{a_1 x_1 + a_2 x_2}{a_1 + a_2}$$

$$\bar{y} = \frac{(150\times t)\times 150 + (t\times 140)\times 70}{(150\times t) + (140\times t)} = 97.9$$

Total length of Weld = $150 + 2(140) = 430\ mm$.

$$I_{xx} = \frac{bd^3}{12} + Ah_1^2$$

$$= \left[\left[\cancel{\frac{150\times t^3}{12}}^{\,\approx 0}\right] + [150\times t]\times(52.1)^2\right] + \left[\frac{t\times(140)^3}{12} + (t\times 140)\times(97.9-75)^2\right]$$

$I_{xx} = 1.08\times10^6 t\ mm^4$

Factored load = $1.5 \times 150 = 225\ KN$

$M = P\times e = 225\times10^3\times 80 = 18\times10^6$.

$$F_1 = \frac{P}{L\times t} = \frac{225\times10^3}{430\times t} = \frac{523.25}{t}$$

$$F_2 = \frac{M}{I_{xx}}\times Y_{max} = \frac{18\times10^6}{1.08\times10^6\times t}\times 97.9 = \frac{1.631\times10^3}{t}$$

$$F_R = \sqrt{\left[\frac{523.25}{t}\right]^2 + \left[\frac{1.631\times10^3}{t}\right]^2} = \frac{410}{\sqrt{3}\times 1.25}$$

$t = 9.04\ mm$

$t = 0.7\times S \Rightarrow S = 12.9\ mm$ say $13\ mm$.

Compression Member :

(A) Angle Strut :

Strut is a inclined member subjected compression load.

Effective Length of Strut :

(i) Angle with Single Bolt → $\boxed{l_e = l}$

(ii) Angle with More Bolts → $\boxed{l_e = 0.85l}$

(iii) Angle with Welding → $\boxed{l_e = 0.70l}$

Radius of Gyration :

$$\boxed{r = \sqrt{\frac{I}{A}}}$$

$I \rightarrow$ M.I.

$A \rightarrow$ Total area.

Slenderness Ratio (λ)

$$\boxed{\lambda = \frac{l_e}{r_{min}}} \ngtr 180$$

Design Compressive Strength $\Big\}$ = $\boxed{P_d = A_e \cdot f_{cd}}$ → Page 34

A_g = Gross area.

f_{cd} = design compressive stress (9(a) - 9(d))

depending on Buckling class

Buckling Class :-

Refer Table 10 Page 44

Eg:- 1

Determine the Compressive Strength of angle strut ISA 100×65×8 mm with a length 3 mts when Connected by

(i) with Single bolt

(ii) More than two bolts.

(iii) welded Connection.

$\boxed{l = 3m}$

Take $f_y = 250$

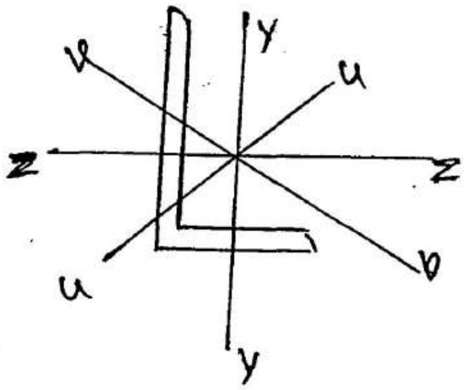
Sol^n (i) Angle with "Single Bolt" :

$\boxed{l_e = l} = 3000mm.$

From Steel Table

ISA 100×65×8mm

C/s area = $12.57 cm^2$



Radius of gyration

$r_{xx} = 3.16cm$ | $r_u = 3.38 cm$

$r_{yy} = 1.83cm$ | $r_v = 1.39cm$

$\therefore \boxed{r_{min} = 13.9mm}$

$\therefore \lambda = \frac{l_e}{r_{min}} = \frac{3000}{13.9} = \boxed{215.83}$

According to Table (10) Page (44)

Single Angle $\longrightarrow$ Buckling class = "c"

$\therefore$ Refer Table 9(c)

Design compressive stress $f_{cd} = 31.72 \; N/mm^2$

$\therefore$ Design Compressive Strength $\Big\} = P_d = (A_e) f_{cd}$

$P_d = \underset{mm^2}{(1257)}(31.72) = \boxed{39.87 \; kN}$

(ii) "More Bolts" are Used :

$\boxed{l_e = 0.85 l} = 0.85 \times 3 = 2550 mm$

$\lambda = \dfrac{l_e}{r_{min}} = \dfrac{2550}{13.9} = \boxed{183.45}$

from Table 9(c) $\rightarrow$ $f_{cd} = 42.25 \; N/mm^2$

$\therefore P_d = (1257)(42.25) = \boxed{53.10 \; kN}$

iii) Welded Connection :

$\boxed{l_e = 0.7 l} = 0.7 \times 3000 = 2100 mm$

$\lambda = \dfrac{l_e}{r_{min}} = \dfrac{2100}{13.9} = \boxed{151.07}$

$\therefore f_{cd} = 58.56 \; N/mm^2$ (Table 9c)

$\therefore P_d = (1257)(58.56) = \boxed{73.61 \; kN}$

Eg:- 2

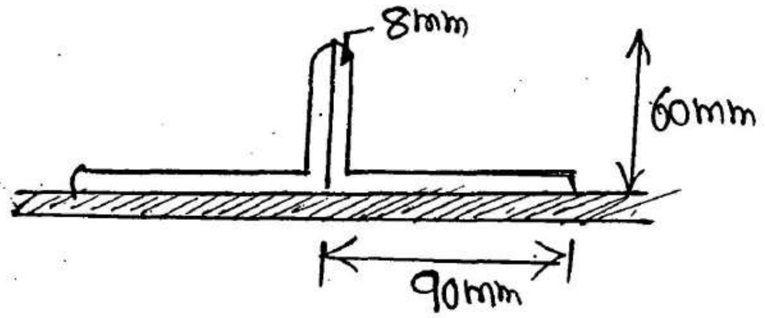
Determine compressive strength of double angle strut 2ISA 90x60x8 mm connected to gusset plate

(i) on same side

(ii) on Both sides

The thickness of gusset plate is 10mm and length is 2.5 mts.

Sol^n (a) Double angle on "Same Side" of G.Plate



$\boxed{l_e = 0.85 l}$

End condition is not given. Assume

$l_e = 0.85 \times 2500 = 2125 mm.$

C/s area $= 22.74 cm^2 = 2274 mm^2$ (2ISA 90x60x8mm)

For double angle → Shorter legs Back to Back

$\gamma_{xx} = \gamma_{zz} = 1.69 cm$

$\gamma_{yy} = 4.10 cm$ (For back to back gap = 0).

$\therefore \boxed{\gamma_{min} = 16.9 mm}$

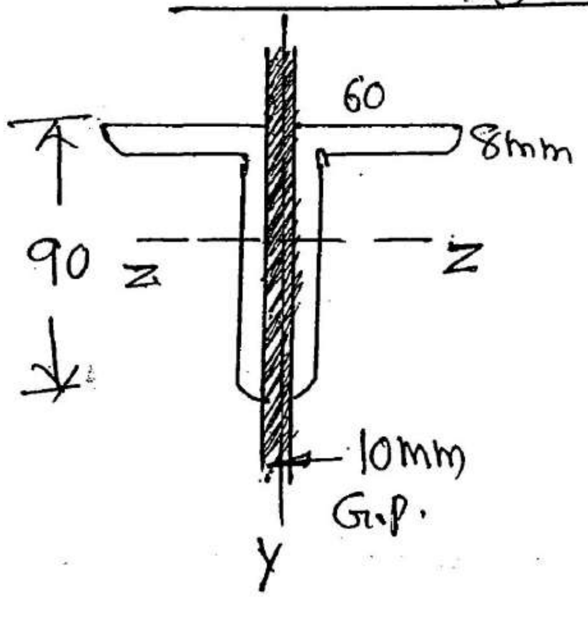
$$\lambda = \frac{l_e}{r_{min}} = \frac{2125}{16.9} = \boxed{125.74}$$

$\therefore$ Buckling class For Built up section $\rightarrow$ "c"
(Table 10 Page 44)

$\therefore$ Refer Table 9(c) $\rightarrow f_{cd} = 78.3\,N/mm^2$

$$\therefore P_d = A_e \cdot f_{cd} = (2274)(78.3)$$
$$= 178.05\,kN$$

(b) Double angle on "Both side of G.P":



Longer leg Back to Back

$r_{xx} = r_{zz} = 2.84\,cm$

$r_{yy} = 2.60\,cm$ (For a gap 1cm)

$\therefore r_{min} = 26\,mm$ $\quad \therefore \lambda = \frac{2125}{26} = \boxed{81.73}$

$\therefore f_{cd} = 133.40\,N/mm^2 \rightarrow$ Table 9c.

$$\therefore P_d = (2274)(133.40) = 303.35\,kN$$

Type-II Eg:- 3] Design a "angle strut" using single angle section to carry a load of 150kN. Use M20 Property class 5.6 Bolts. the length of the member is 2.5m.

Sol^n

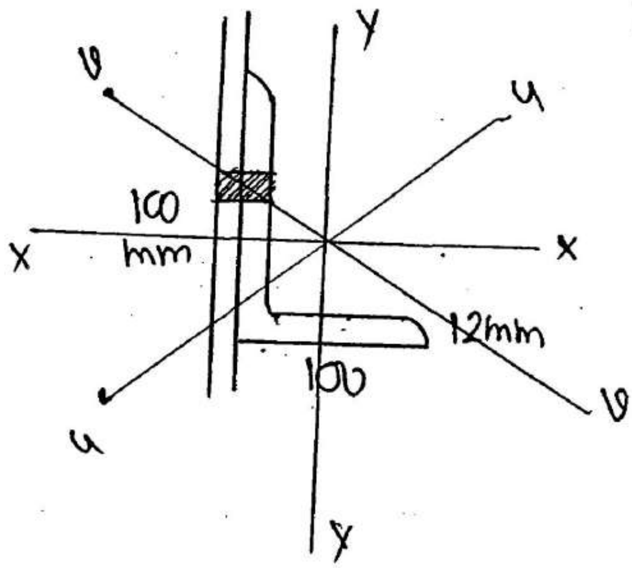
Factored load = $1.5 \times 150 = 225\,kN$.

(a) Assume $\boxed{f_{cd} = 100\ N/mm^2}$

$$\text{Area})_{Req} = \frac{\text{Load}}{f_{cd}} = \frac{225 \times 10^3}{100} = 2250\,mm^2 = 22.50\,cm^2.$$

From steel Table Try ISA 100x100x12mm

(area = 22.59 cm^2)



From steel Table

$$\left.\begin{aligned} r_{xx} &= 3.03\,cm \\ r_{yy} &= 3.03\,cm \\ r_{uu} &= 3.82\,cm \\ r_{vv} &= 1.94\,cm \end{aligned}\right\} \therefore r_{min} = 19.4\,mm$$

Effective length $\boxed{l_e = 0.85\,l} = 0.85 \times 2.5 = 2.125\,m$

$$\lambda = \frac{l_e}{r_{min}} = \frac{2125}{19.4} = \boxed{109.54}$$

For single angle Buckling class — "C"

(Table 10 Page 44)

$$\therefore \boxed{f_{cd} = 94.6 \text{ N/mm}^2} \longrightarrow \text{from Table 9(c)}$$

$$\therefore \left.\begin{array}{l}\text{Design Compressive}\\ \text{Strength}\end{array}\right\} = \boxed{P = A_e \cdot f_{cd}}$$

$$P = (2259)(94.6) = 213.70 \text{ kN} < 225 \text{ kN}$$

(Un-Safe)

Hence Revise the section

Now Try ISA $\cancel{150}$ 125 × 95 × 12 mm

$$\therefore \text{area} = \cancel{2862}\ 2498 \text{ mm}^2$$

$$r_{min} = \cancel{1.58}\ 2.01 \text{ cm} = \cancel{15.8}\ 20.1 \text{ mm}$$

$$\lambda = \frac{l_e}{r_{min}} = \frac{2125}{\cancel{15.8}\ 20.1} = \boxed{\cancel{134.8}}\ \boxed{105.7}$$

$$\therefore f_{cd} = \cancel{101.17}\ 99.93 \text{ N/mm}^2$$

$\therefore\ P = A_e \cdot f_{cd} = (2498)(99.93)$

$P = 249.62\,kN > 225\,kN$ (Safe).

(b) <u>Connections</u> : M_{20} Property class 5.6

(i) <u>In shear</u> :

$$V_{nsb} = \frac{500}{\sqrt{3}}\left(1 \times 0.78 \times \frac{\pi}{4}(20)^2 + 0\right) = 70.74\,kN$$

$$\therefore\ V_{dsb} = \frac{70.74}{1.25} = \boxed{56.59\,kN}$$

(ii) <u>In bearing</u> :

$e = 1.7 \times 22 \approx 40\,mm$

$p = 2.5 \times 20 = 50\,mm$

$k_b \rightarrow$ (i) 0.606 (ii) 0.507

(iii) 1.22 (iv) 1.0

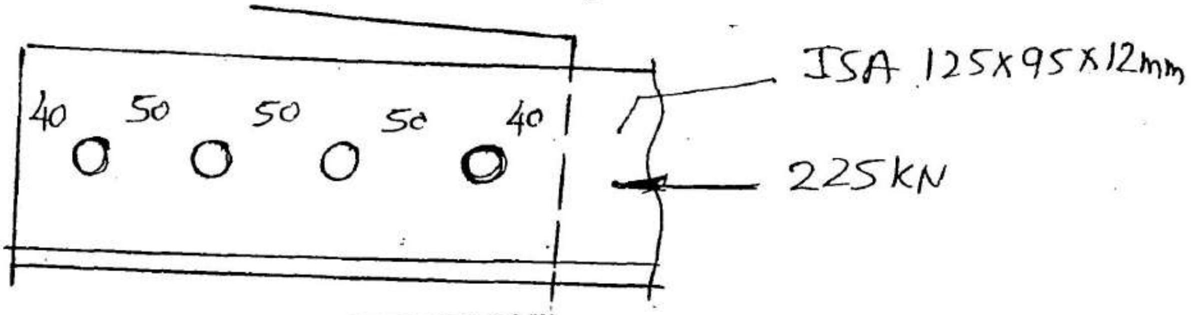
$\therefore\ \boxed{k_b = 0.507}$

$\therefore\ V_{npb} = 2.5 \times 0.507 \times 20 \times 12 \times 410 = 124.72\,kN$

$\therefore\ V_{dpb} = \boxed{99.78\,kN}$

$\therefore$ Bolt Value $= 56.59\,kN$

$$\therefore\ \text{No. of Bolts} = \frac{225}{56.59} \approx 4$$



Eg:- $\boxed{2}$ Design a angle strut using double angle to carry a load 400kN. Use Welded Connection. Take length of the member 2m.

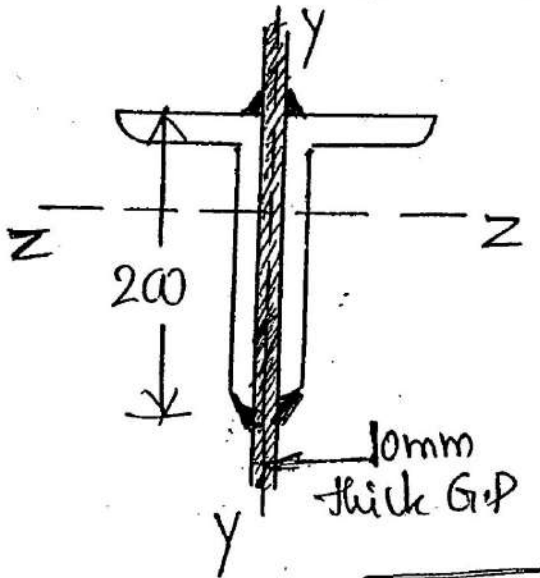
10/12

Factored load $= 1.5 \times 400 = 600\,kN$

(a) Assume $f_{cd} = 120\,N/mm^2$

$$Area)_{Req} = \frac{Load}{f_{cd}} = \frac{600 \times 10^3}{120} = 5000\,mm^2 = 50\,cm^2$$

From steel Table Try 2 ISA 200 X 100 X 10mm



area $= 5806\,mm^2$

$r_x = 6.46\,cm$

$r_y = 3.68\,cm$

$\therefore r_{min} = 36.8\,mm$

$\boxed{l_e = 0.7 \times l} \rightarrow$ Welded Connection

$$\lambda = \frac{l_e}{r_{min}} = \frac{0.7 \times 2000}{36.8} = \boxed{38.04}$$

For Built up section → Buckling class "c"

(Table 10)

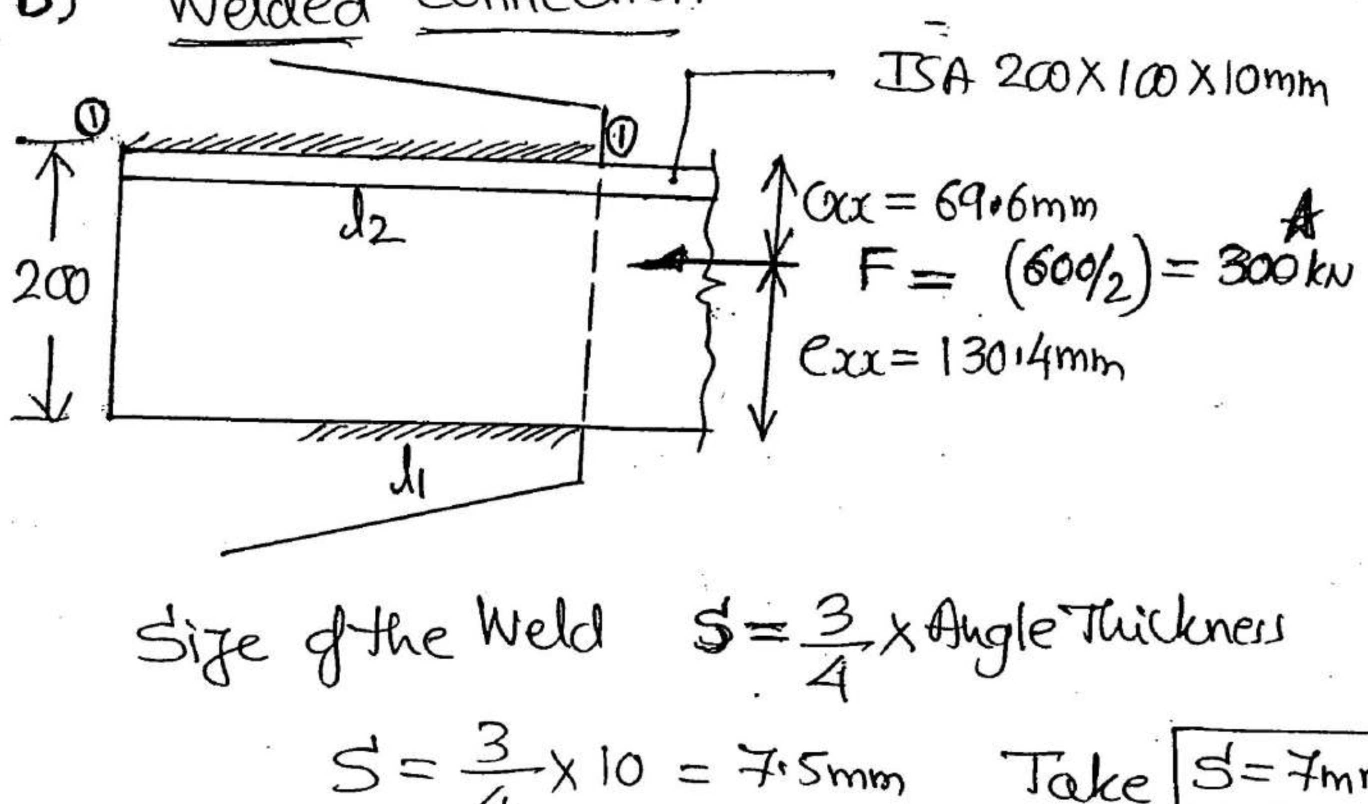
∴ Use Table 9(c) → $f_{cd} = 200.55\ N/mm^2$

∴ Compressive strength $P = A_e \cdot f_{cd}$.

$$P = (5806)(200.55) = \boxed{1164.3\ kN} > 600\ kN$$

(Over safe).

(b) Welded Connection



Size of the Weld $S = \frac{3}{4} \times$ Angle Thickness

$$S = \frac{3}{4} \times 10 = 7.5mm \quad \text{Take } \boxed{S = 7mm}$$

Force = Strength of the Weld.

$$300 \times 10^3 = (0.7 \times 7)(l)\left(\frac{410}{\sqrt{3} \times 1.25}\right)$$

$$\therefore l = 323.30mm = l_1 + l_2 \quad \text{—(i)}$$

Taking moment about 1-1

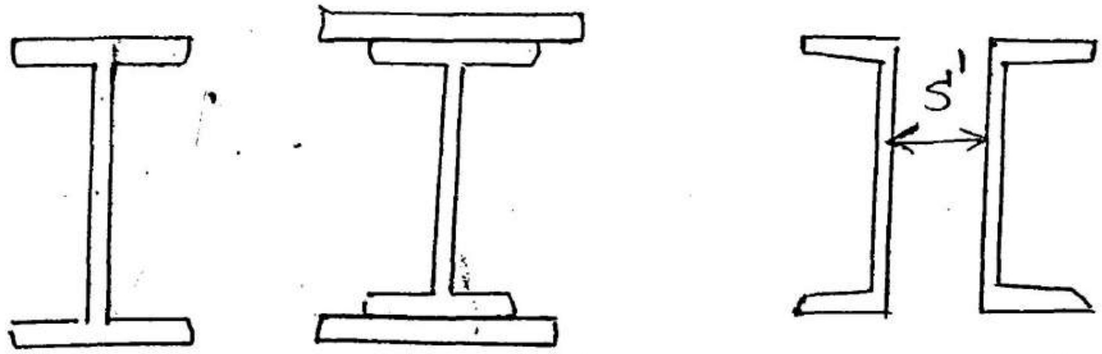
$$(300 \times 10^3) \times 69.6 = \left[0.7 \times 7 \times l_1 \times \frac{410}{\sqrt{3} \times 1.25}\right] 200$$

$$\therefore l_1 = \underline{\underline{112.51mm}}$$

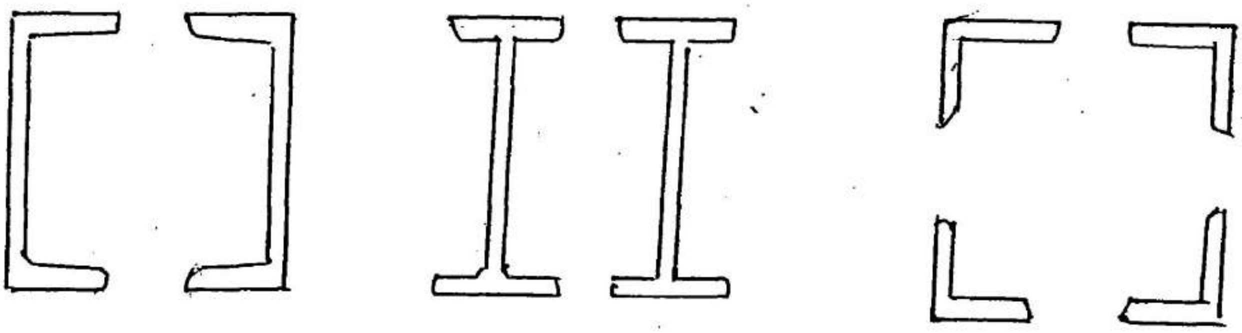
$$\& \ l_2 = 323.3 - 112.5 = 210.78 mm$$

$$\boxed{\text{Provide } l_1 = 115mm, \ l_2 = 215mm}$$

: Column Design :

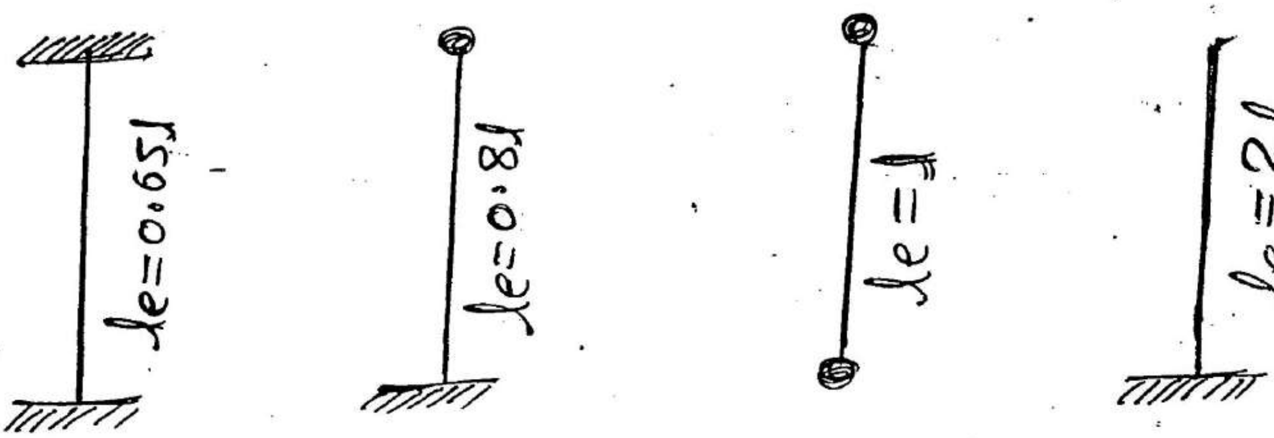


Double channel Back to Back (Heel to Heel)



Double channel Face to Face (Toe to Toe)

Effective length (Page 45)



$$\left.\begin{array}{c}\text{Design Compressive}\\ \text{Strength}\end{array}\right\} = \boxed{P = Ae \cdot fcd}$$

Eg:- 1

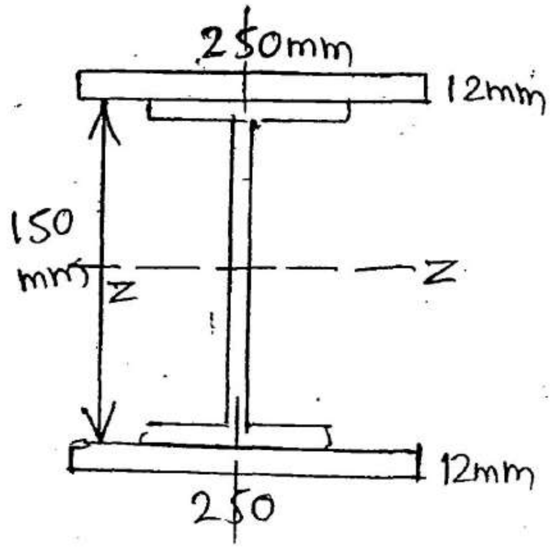
Design a Column section using Single rolled steel beam along with Cover Plates to Carry a factored load of 2000kN. The Column both ends are fixed. Take $l = 6m$.

(a) Assume $f_{cd} = 220 N/mm^2$

$$Area)_{Req} = \frac{Load}{f_{cd}} = \frac{2000 \times 10^3}{220} = 9090.9 mm^2$$
$$= 90.90 cm^2$$

From Steel Table → Rolled Steel Beams with Cover plates

Try ISHB 150 @ 34.6 kg/m and Cover plate 250mm × 12mm



$area = 104.08 \times 100 mm^2$

$r_x = r_z = 7.32 cm$

$r_y = 5.90 cm$

$\therefore r_{min} = 59 mm$

$\boxed{l_e = 0.65 l}$ Given Both ends fixed

$= 0.65 \times 6000 = 3900 \text{ mm.}$

$\therefore \lambda = \frac{l_e}{r_{min}} = \frac{3900}{59} = \boxed{66.10}$

Buckling Class → © (∵ Built up section)

∴ Table 9(c) → $\boxed{f_{cd} = 158.24}$ N/mm².

∴ Design compressive strength $\Big\} = P = A_e \cdot f_{cd}$

$P = (104.08 \times 100)(158.24) = 1647.0 \text{ kN}$

$< 2000 \text{ kN}$

(Un-Safe)

Hence "Revise" the Section

Now Try ISHB-150 @ 30.6 kg/m

& 250mm x 20mm cover plate

∴ area $= 138.98 \times 100 \text{ mm}^2$

$r_x = r_z = 7.96 \text{ cm}$

$r_y = 6.39 \text{ cm}$ $\Big\}$ ∴ $r_{min} = 63.9 \text{ mm}$

$\therefore \lambda = \frac{l_e}{r_{min}} = \frac{3900}{63.9} = \boxed{61.0}$

$f_{cd} = 166.4\ N/mm^2$

$\therefore$ Design Comp. Str. $= P = A_e \cdot f_{cd}$

$P = (13898)(166.4) = 2312.6\ kN > 2000\ kN$

(Safe)

$=\!=\!= \times =\!=\!=$

Eg:- 2

Design a compression member using double channel section back to back to carry a factored load of 1600 kN. The length of the column is 5 m with one end fixed & one end hinged.

Sol:

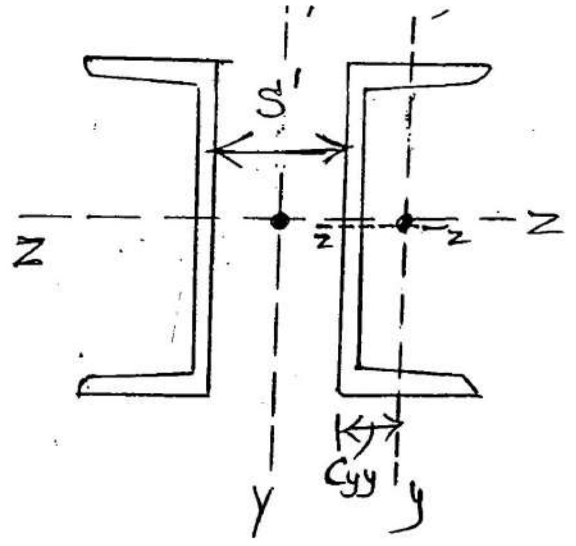
(a) Assume $f_{cd} = 180\ N/mm^2$.

$$\text{Area})_{Req} = \frac{\text{Load}}{f_{cd}} = \frac{1600 \times 10^3}{180} = 8888.8\ mm^2 = 88.88\ cm^2.$$

From steel Table select double channel back to back

Try 2 ISLC - 350 @ 77.6 kg/m

(Area = $98.94\ cm^2$).



Properties of one channel

ISLC - 350 @ 38.8 kg/m

area = $4947\ mm^2$

$I_x = 9312.6 \times 10^4$

$I_y = 394.6 \times 10^4$

$C_{yy} = 24.1\ mm$

The spacing between two channels should be such that

$$\boxed{I_{xx} = I_{yy}}$$

$$\left(\begin{aligned} I_{xx} &= I_{zz} \\ I_{xx} &= I_{zz} \end{aligned} \right)$$

$$I_{xx} = I_{xx} + ah^2$$

$$I_{xx} = 2\left[9312.6 \times 10^4 + (4947)(0)\right] = 18625.2 \times 10^4\ mm^4$$

$$I_{yy} = 2\left[394.6 \times 10^4 + (4947)\left(C_{yy} + \frac{S'}{2}\right)^2\right]$$

$$= 2\left[394.6 \times 10^4 + (4947)\left(24.1 + \frac{S'}{2}\right)^2\right]$$

Equating I_x & I_y

$$18625.2 \times 10^4 = 2\left[394.6 \times 10^4 + 4947\left(24.1 + \frac{S'}{2}\right)^2\right]$$

$$\therefore \boxed{S' = 220.35}\ mm$$

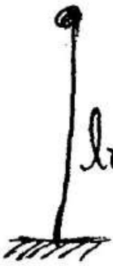
Or Approximately $\rightarrow$ $I_{xx} \approx I_{yy}$

$\therefore$ Spacing $\boxed{S' = 220mm}$

$\therefore I_{min} = 18625.2 \times 10^4$

$$\gamma_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{18625.2 \times 10^4}{2 \times 4947}} = 137.2 mm$$

$$\lambda = \frac{l_e}{\gamma_{min}} = \frac{0.8 \times 5000}{137.2} = \boxed{29.15}$$



$\therefore$ For Built up section $\longrightarrow$

Buckling class (c) $\therefore$ Table 9(c)

$f_{cd} = 211.0 N/mm^2$

$$\therefore P = (2 \times 4947) 211 = 2087.63 kN$$

$> 1600 kN$ (Safe).

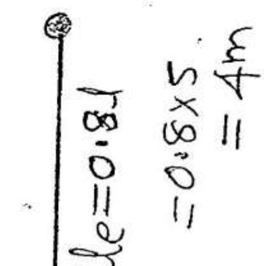
—— x ——

3]

Design a compression member using double channel section "Face to Face" to carry a Factored load of 1600 kN. The length of the column is 5m. with one end fixed & one end hinged.

Also design "single lacing system"

(A) Design of "Column"

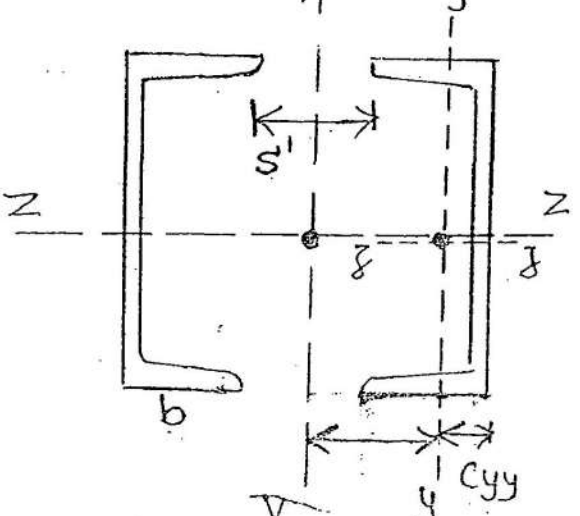


Assume $f_{cd} = 200 N/mm^2$

$$Area)_{Req} = \frac{load}{f_{cd}} = \frac{1600 \times 10^3}{200} = 8000 mm^2 = 80 cm^2$$

$$\text{area for one channel} = \frac{80}{2} = 40 cm^2$$

From Steel Table Try 2 ISLC-300 @ 33.1 kg/m



Properties of one channel

area = 4211 mm^2

$I_{xx} = 6047.9 \times 10^4$

$I_{yy} = 346.0 \times 10^4$

$C_{yy} = 25.5 mm$

$b = 100 mm$

The spacing $\underline{S'}$ should be such that

$$\boxed{I_{xx} = I_{yy}}$$

$$I_{xx} = 2\left[6047.9 \times 10^4 + 4211(0)^2\right] = 120.958 \times 10^6 \text{ mm}^4$$

$$I_{yy} = 2\left[346 \times 10^4 + 4211\left(b - C_{yy} + \frac{S'}{2}\right)^2\right]$$

$$\therefore 120.958 \times 10^6 = 2\left[346 \times 10^4 + 4211\left(100 - 25.5 + \frac{S'}{2}\right)^2\right]$$

$$\therefore \boxed{S' = 83.73} \text{ mm}$$

$$\therefore I_{min} = 120.958 \times 10^6 \text{ mm}^4$$

$$r_{min} = \sqrt{\frac{I_{min}}{A}} = \sqrt{\frac{120.958 \times 10^6}{2 \times 4211}} = \boxed{119.84}$$

$$\therefore \lambda = \frac{l_e}{r_{min}} = \frac{4000}{119.84} = 33.38$$

$\therefore$ Table 9(c) $\longrightarrow$ Buckling class (c)

$$\boxed{f_{cd} = 206.61 \text{ N/mm}^2}$$

$\therefore$ Comp. load $P = A_e \cdot f_{cd}$

$$P = 2 \times 4211 \times 206.61$$

$$\boxed{P = 1740.0 \times 10^3 \text{ N}} > 1600 \text{ kN}$$

(Safe).

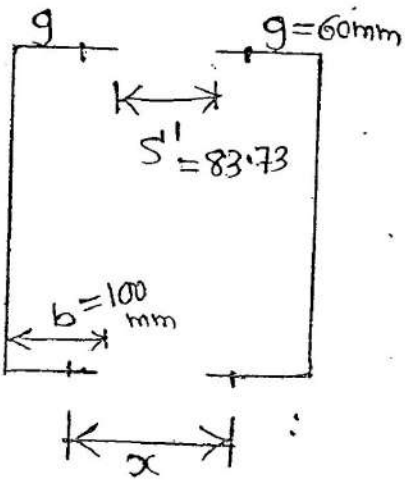
(B) Design of Lacing : (Page 48-49-50)

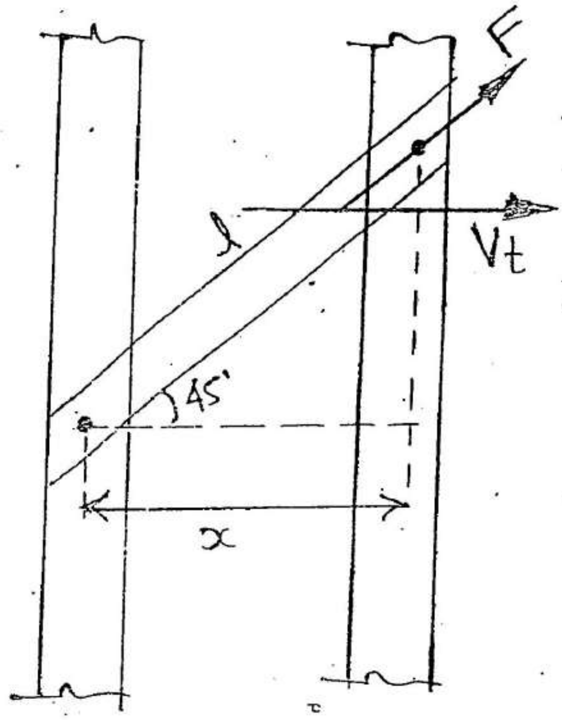
(1) Transverse shear = 2.5% of column Load

$$V_t = \frac{2.5}{100} \times 1600 = \boxed{40\text{ kN}}$$

(2) Lacing Inclination = $\boxed{\theta = 45^\circ}$

(3) Length of lacing





$$x = 2(b-g)+S'$$

$$\bar{x} = 2(100-60)+83.73$$

$$\boxed{x = 163.73\text{mm}}$$

$$\therefore \cos 45^\circ = \frac{x}{l} \qquad \therefore\ l = \frac{163.73}{\cos 45} = \boxed{231.55\text{mm}}$$

For single lacing $(l_e = l) = 231.55\text{mm}$

Double lacing $(l_e = 0.7\,l)$.

(4) Lacing dimensions (b & t) :

width = $\boxed{b = 3 \times \text{dia. of Bolt}}$ Assume 18mm ϕ Bolt

$\therefore b = 3 \times 18 = 54mm$ Take $\boxed{b = 55mm}$

Thickness $\boxed{t = \frac{le}{40}}$ → single lacing

* $\boxed{t = \frac{le}{60}}$ → For double lacing

$\therefore t = \frac{le}{40} = \frac{231.55}{40} = 5.79mm \approx 6mm$

$\therefore$ Lacing Bar → $\boxed{b \times t = 55mm \times 6mm}$

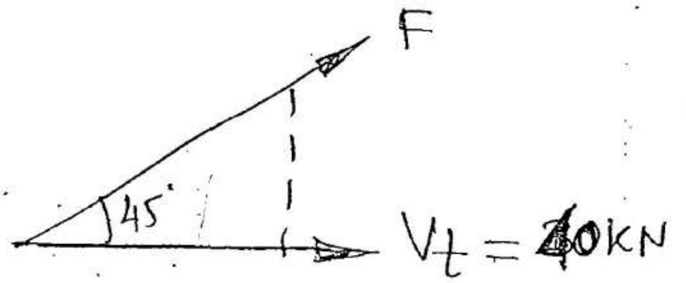
(5) Check for slenderness Ratio :

$\boxed{\lambda = \frac{le\sqrt{12}}{t}} \not> 145.$

$= \frac{231.55\sqrt{12}}{6} = 133.68 < 145$ (Safe)

$\therefore \boxed{f_{cd} = 71.3 N/mm^2}$ (Table 9c)

(6) Force in lacing (F).



$$\cos 45 = \frac{V_t}{F}$$

$$\therefore \boxed{F = \frac{V_t}{n \cdot \cos\theta}}$$

* n → No. of planes of lacing = 2

$$\therefore F = \frac{40 \times 10^3}{(2)\cos 45^\circ}$$

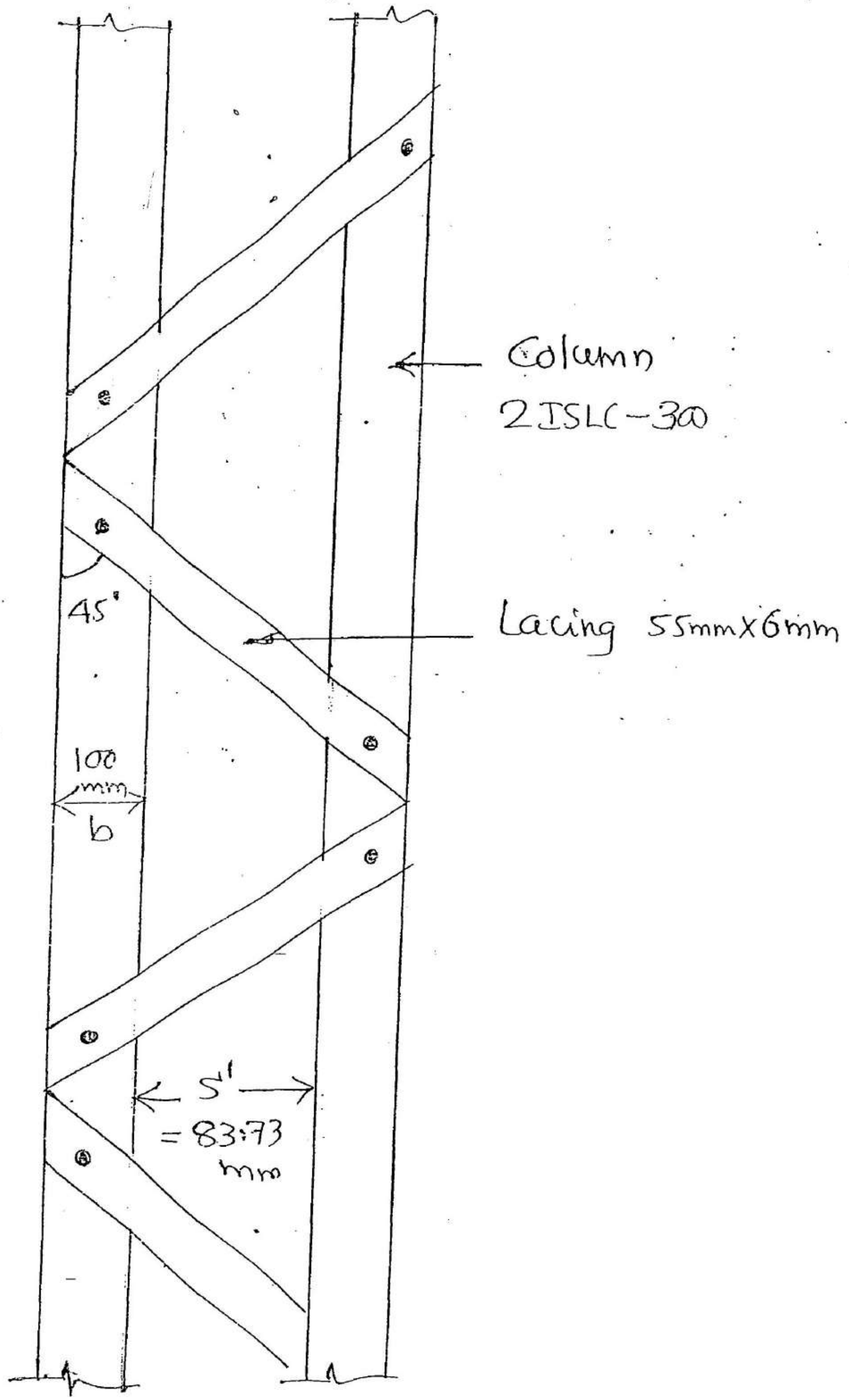
$$\boxed{F = 28.28\,kN}$$

(7) Check for strength :

(i) Comp. strength = $b \times t \times f_{cd} = 55 \times 6 \times 71.3$
$= 23.5 \times 10^3\,N$

(ii) Tensile strength = $(b - d_o)\,t \times f_{cd}$ =
$= (55 - 20) \times 6 \times 71.3 = 14.97 \times 10^3\,N$

(8) Provide "single Bolt" at each end.

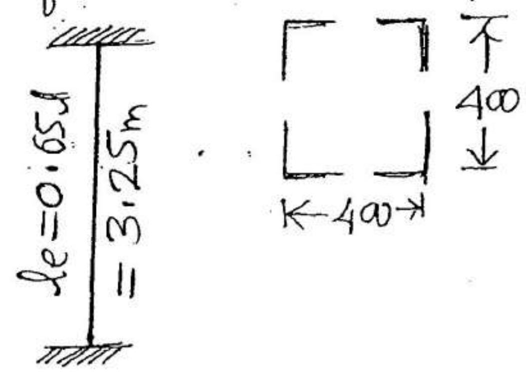

Column
2ISLC-300
Lacing 55mmX6mm
45°
100 mm
b
S' = 83.73 mm

Eg:- 4

Design a Column section consisting of "4 angle" sections arranged in a box shape 400 mm × 400 mm to carry an axial load of 2500 kN. The height of the column is 5 mts and both ends are fixed.

Sol:

$P_u = 2500\,kN.$

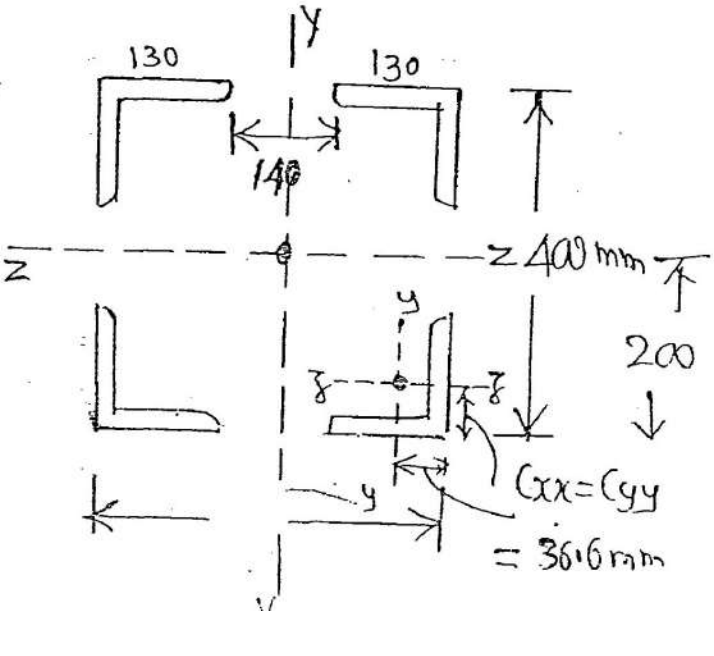


(a) Assume $f_{cd} = 220\ N/mm^2$

$$\therefore \text{Area} = \frac{2500 \times 10^3}{220} = 11363.64\,mm^2 = 113.63\,cm^2$$

$$\therefore \text{Area for } \underline{\text{one}} \text{ angle} = \frac{113.63}{4} = 28.41\ cm^2.$$

From steel Table 4 ISA 130 × 130 × 12mm



Properties of "one" angle

area = $2982\ mm^2$

$I_x = I_y = 473.8 \times 10^4$

$C_x = C_y = 36.6\,mm$

$$I_z = I_y = I_{min} = 4\left[473.8\times10^4 + 2982(200-26.6)^2\right]$$

$$= 33\cancel{7}4\times10^6$$

$$r_{min} = \sqrt{\frac{337.4\times10^6}{4\times2982}} = 168.2\ \text{mm}$$

$$\lambda = \frac{l_e}{r_{min}} = \frac{3.25\times10^3}{168.2} = 19.32$$

Table 9(c) $\therefore f_{cd} = 224\ \text{N/mm}^2$

$$\therefore P = A_e \cdot f_{cd}$$

$$P = 4\times2982\times224 = 2672\times10^3\ \text{N}$$

$$> 2500\ \text{kN (safe)}$$

—x—

Column Splices :-

(A) Same Column Size

Design Specification :

1. Flange splice is designed for load & moment.
   Web splice is designed for horizontal shear.
2. If column faces machined or milled or grinded then flange splice is designed for 50% of column load.
3. Width of splice plate = column flange width
4. If moment is acting on the column it is converted into addional load $= \frac{M}{h}$
5. If column sizes are different then provide bearing plate in between.

Eg:- 1]

Design a "Column splice" for ISHB 350 @ 72.4 subjected to 600 kN load, 50 kN-m moment and 150 kN horizontal shear. use M20 property class 5.6 bolts

Design both flange splice & web splice.

Solⁿ:

ISHB-350 @ 72.4 kg/m → $h = 350$, $b = 250$
$t_f = 11.6$, $t_w = 10.1$

$\left.\begin{array}{l}\text{Load} = 600\,kN \\ \text{Moment} = 50\,kN\text{-}m\end{array}\right\} \rightarrow$ Flange splice

Shear = 150 kN → Web splice

(a) Design of "Flange Splice":

$$\text{Area of Flange splice} = \frac{\text{Load}}{\text{Stress}}$$

★ Assume column faces are machined or milled or ~~grinded~~ grinded

$$\therefore \text{Load on flanges} = \frac{600}{2} = 300\,kN$$

$$\text{Load on each flange} = \frac{300}{2} = \boxed{150\,kN}$$

$$\text{Convert Moment into additional load} = \frac{M}{h}$$

$$= \frac{50\,kN\text{-}m}{h = 0.35\,m} = \boxed{142.86\,kN}$$

$\therefore$ Total load for Flange splice $\Big\}$ = 292.86 kN

$$\therefore \text{Area} = b \times t = \frac{292.86 \times 10^3}{\left(\frac{250}{1.10}\right)} = 1288.58\,mm^2$$

$$\left(\frac{f_y}{\gamma_{m0}}\right)$$

Provide splice width = Flange width = 250mm

$$\therefore t = \frac{1288.58}{250} = 5.15\,mm \approx \underline{6\,mm}$$

$\therefore$ $\boxed{\text{Flange splice} \rightarrow 250mm \times 6mm}$

(b) <u>Design of "Web splice"</u> :-

Horizontal shear = 150 kN

$$\therefore \text{Area} = \frac{\text{Load}}{\text{Stress}} = \frac{150 \times 10^3}{\left(\frac{250}{1.10}\right)} = 660\,mm^2$$

Using 6mm thick plate $\therefore b = \frac{660}{6} = 110mm$

$\therefore$ $\boxed{\text{Web splice} \rightarrow 110mm \times 6mm}$

(c) Connection :

$M_{20} \rightarrow$ Property class 5.6

(i) Shear :

$$V_{dsb} = \frac{1}{1.25}\left[\frac{500}{\sqrt{3}}\left(1 \times 0.78 \times \frac{\pi}{4}(20)^2 + 0\right)\right]$$

$$= \boxed{56.6 \text{ kN}}$$

(ii) Bearing : $e = 40\text{mm}$, $p = 50\text{mm}$

$k_b \rightarrow$ (i) 0.606 (ii) $\boxed{0.507}$ (iii) 1.22 (iv) 1.0

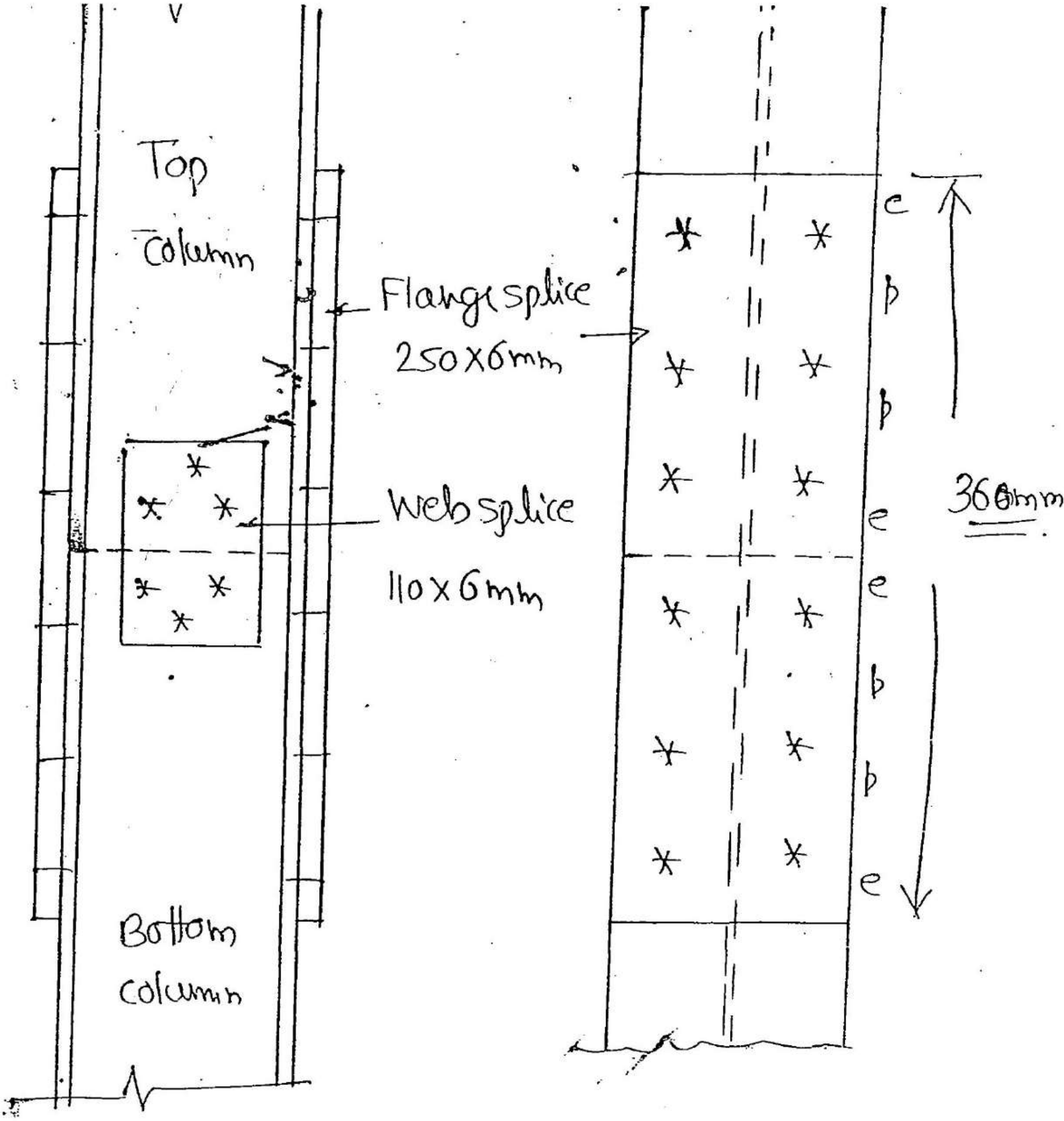
$$V_{dpb} = \frac{1}{1.25}\left[2.5 \times 0.507 \times 20 \times 6 \times 410\right] =$$

$$= \boxed{49.88}$$

$\therefore$ Bolt Value = 49.88 kN

No. of Bolt for Flange Splice $= \dfrac{298.86}{49.88} \approx 6$

No. of Bolts for Web Splice $= \dfrac{150}{49.88} \approx 3$



Top
column
Flange splice
250 X 6mm
Web splice
110 X 6mm
Bottom
column
e
p
p
e
360mm
e
p
p
e

Column Splices with Different size

(1) Design a column splice for a column ISHB 300 @ 58.8 kg/m supported on ISHB 350 @ 67.4 kg/m. The load on the column is 800 kN and moment 50 kN/m
Use M20 HSFG property class 8.8 bolts.
Design column splice and bearing plate in between.

50/12

ISHB-300 $\rightarrow$ $h = 300$, $b = 250$, $t_f = 10.6$ mm

ISHB-350 — $h = 350$, $b = 250$, $t_f = 11.6$ mm

Load $= 800$ kN, Moment $= 50$ kN-m.

$\therefore$ Ultimate load $= \boxed{1200\text{ kN}}$ & Ultimate Moment $= \boxed{75\text{ kN-m}}$

Assume Column faces are unmachined.

(a) Design of column splices :

Load on flanges $= 1200$ kN

$\therefore$ Load on each flange $= \frac{1200}{2} = \boxed{600\text{ kN}}$

Additional load due to Moment $= \frac{M}{h} = \frac{75\text{ kN-m}}{0.30\text{m}}$

$= \boxed{250\text{ kN}}$

850 kN

$$\left.\begin{matrix}\text{Area of Flange}\\ \text{splice}\end{matrix}\right\} = b \times t = \frac{\text{Load}}{\left(f_y/\gamma_{m0}\right)} = \frac{850 \times 10^3}{\left(\frac{250}{1.10}\right)}$$

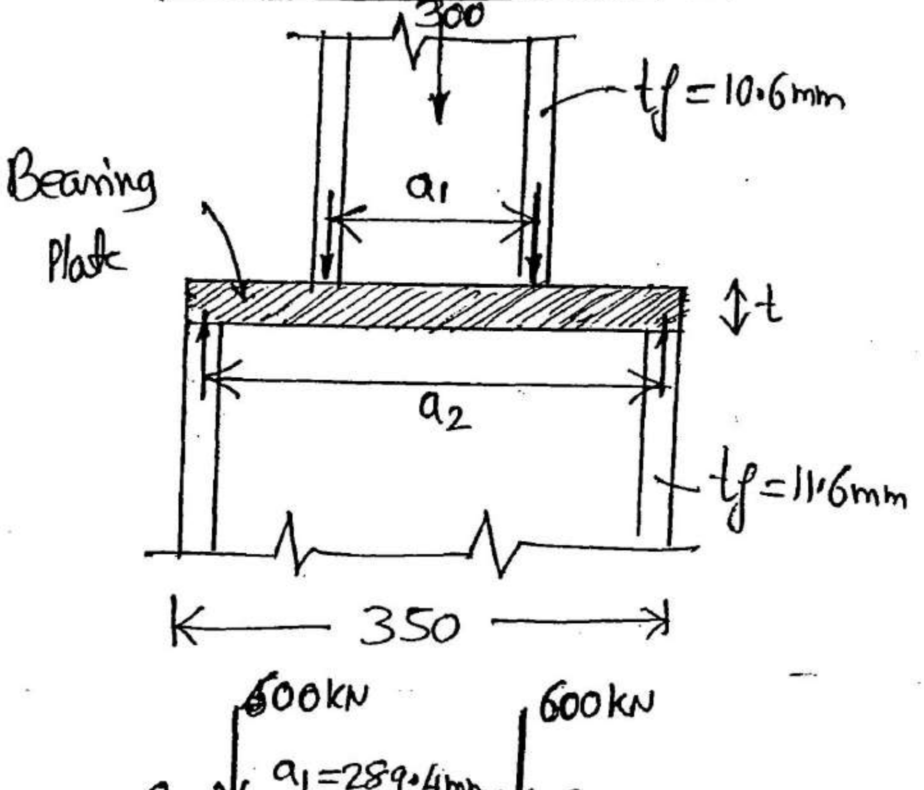
$$b \times t = 3740\,mm^2.$$

Provide width of splice = width of col. flange = 250mm

$$\therefore\ t = \frac{3740}{b=250} = 14.96mm \approx \underline{\underline{16mm}}$$

$$\therefore \text{Flange splice} \rightarrow \boxed{b \times t = 250mm \times 16mm}$$

(b) "Bearing Plate" Design :

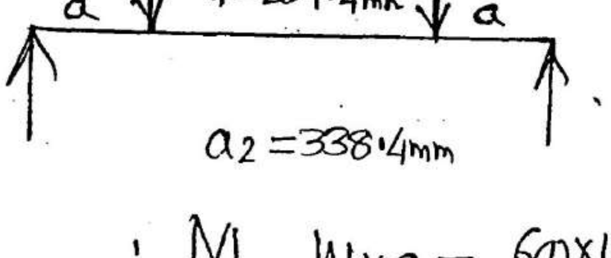


$$a_1 = 300 - 10.6$$
$$= 289.4mm$$

$$a_2 = 350 - 11.6$$
$$= 338.4mm$$

$$\therefore\ a = \frac{a_2 - a_1}{2}$$

$$a = \underline{\underline{24.5mm}}$$



$$\therefore\ M = W \times a = 600 \times 10^3 \times 24.5 = \boxed{14.7 \times 10^6}\ N\text{-}mm$$

Using $\frac{M}{I} = \frac{f}{y}$ $\quad M = \left(\frac{I}{y}\right) f$

$\boxed{M = Z \times \sigma_b}$ $\quad Z = \frac{bd^2}{6}$, $\sigma_b = \left(\frac{f_y}{\gamma_{mo}}\right)$

$14.7 \times 10^6 = \frac{250 \times t^2}{6} \times \left(\frac{250}{1.10}\right)$ $\quad \therefore \boxed{t = 40mm}$

$\therefore$ $\boxed{\text{Bearing Plate} = 250mm \times 350mm \times 40mm}$

(c) Connections :

M_{20} – HSFG P. Class 8.8

$\mu_f = 0.55$, $\quad k = 1.0$, $\quad n_e = 1$,

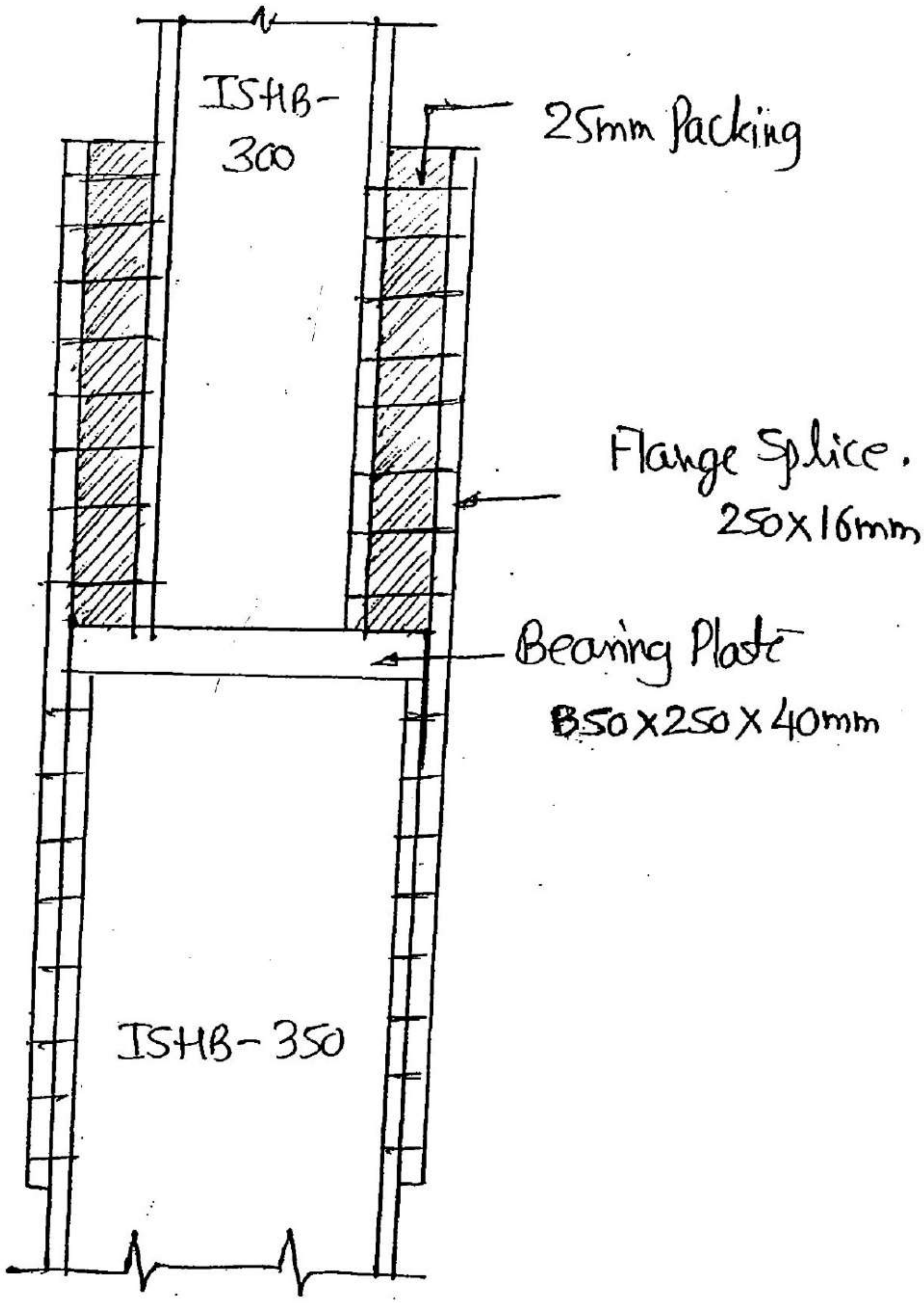
$F_0 = A_{nb} \times 0.7 f_{ub} = 0.78 \times \frac{\pi}{4}(20)^2 \times 0.7 \times 800$

$= 137.22\, kN$

$\therefore V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{1}{1.25}\left[\mu_f \times k_o \times n_e \times F_0\right]$

$= \frac{1}{1.25}\left[0.55 \times 1 \times 1 \times 137.22\right] = 60.38\, kN$

$\therefore$ No. of Bolts $= \frac{\text{Force}}{B.V} = \frac{850}{60.38} \simeq 15 \simeq 16$


ISHB-300
25mm Packing
Flange Splice.
250X16mm
Bearing Plate
B50X250X40mm
ISHB-350

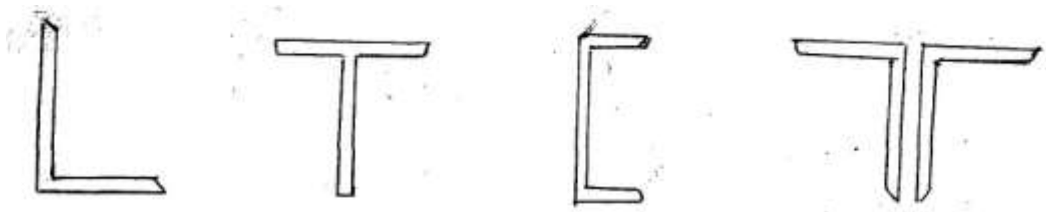
DESIGN OF STEEL STRUCTURAL ELEMENTS (18CV61)

MODULE 4
DESIGN OF TENSION MEMBER

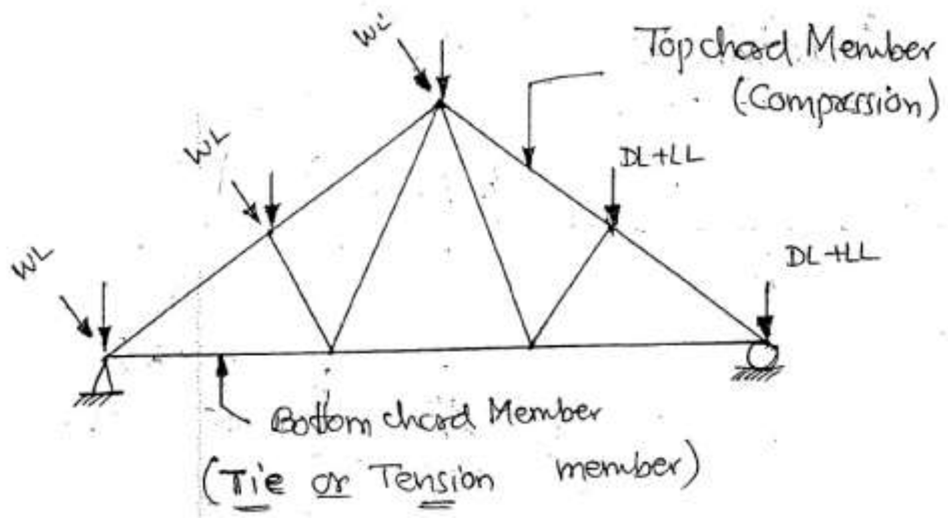
MODULE 04

DESIGN OF TENSION MEMBERS

- ✓ Tension members are linear members in which actual forces act to cause elongation (stretch).
- ✓ Tension members also known as "TIE members"
- ✓ Tension members sustain loads upto the ultimate load, after that they may fail by rupture at critical Section.
- ✓ Following are the types of section used for tension member. i.e. L, T, C, I, Double angle sections etc.



- ✓ A truss is a combination of tension and compression member as shown in the following in figure



Slenderness Ratio: (λ):

"The ratio of effective length of the member to the minimum radius of gyration is called slenderness ratio."

$$\therefore \boxed{\lambda = \frac{l_e}{r_{min}}}$$

$$\boxed{l_e = 0.85\,l}$$

(i) $\boxed{\lambda \ngtr 180}$ → For the "reversal" of stresses due to other than wind load

ii) $\boxed{\lambda \ngtr 350}$ → If the "reversal" stresses due to wind load.

Reversal of Stresses:

Due to change in direction of wind or seismic load, there is a change in nature of stresses in a member and it is called as reversal of stresses.

According to code

λ > not greater than 180 – for reversal of the stresses due to other than wind load and seismic load.

λ > not greater than 350 – for reversal of the stresses due to wind load and seismic load.

Effective Length: (le)

i. For single bolt connection $le = l$

ii. For more bolts $le = 0.85\,l$

iii. For a welded connection $le = 0.7l$

Gusset plate:

- ✓ A gusset plate is a plate provided at the ends of tension members through which forces are transfer to the main member.
- ✓ Gusset plate may be used to join members at a joint and line of action of truss members meeting at a joint should coincide.
- ✓ There is not standard size and shape of gusset plate.

Design Strength of a tension member (T_d)

The design strength of steel tension member is least of the following

1. Design strength due to yielding of C/S (T_{dg})
2. Design strength due to rupture of C/S (T_{dn})
3. Design strength due to block shear (T_{db})

(1) Design strength due to "Yielding"

(Solid strength of Plate)

$$\boxed{T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}}} \longrightarrow \text{Page (32)}$$

$A_g \rightarrow$ Gross area,

$f_y = 250 \, N/mm^2$

$\gamma_{mo} = 1.10$

(2) Design strength due to "Rupture" : (For Plates)

$$\boxed{T_{dn} = \frac{0.9 A_n \cdot f_u}{\gamma_{m1}}} \longrightarrow \text{Page (32)}$$

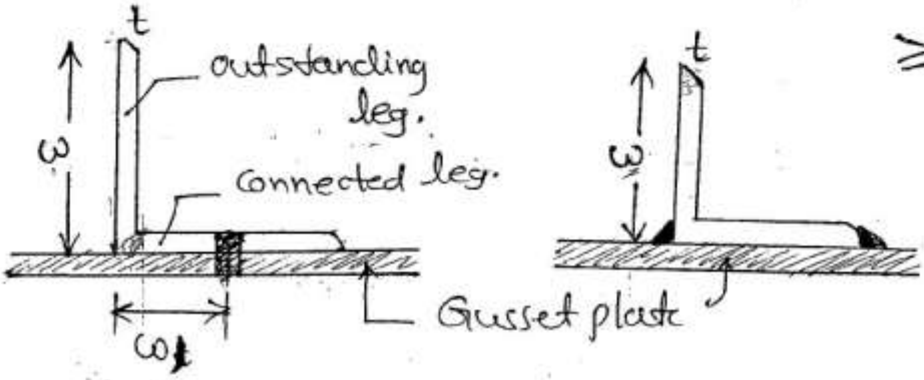
$$A_n = \left[b - n d_h + \sum \frac{p_s^2}{4g} \right] t$$

Rupture strength for "Angle"

$$\boxed{T_{dn} = \frac{0.9\, A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta\, A_{go} \cdot f_y}{\gamma_{mo}}}$$ Page (33)

where $\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \left(\frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}}\right)$

≥ 0.7



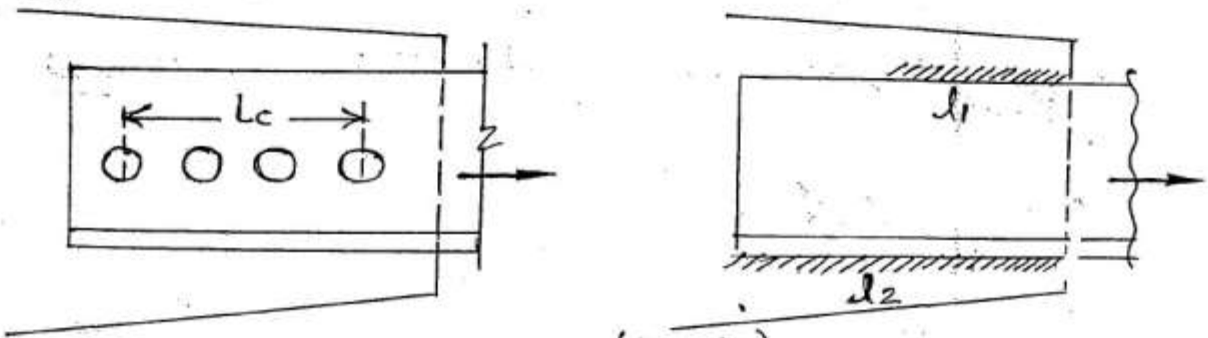
b_s = Shear lag width

$b_s = (w + w_1 - t)$ → For Bolted connection

$b_s = w$ → For Welded connection

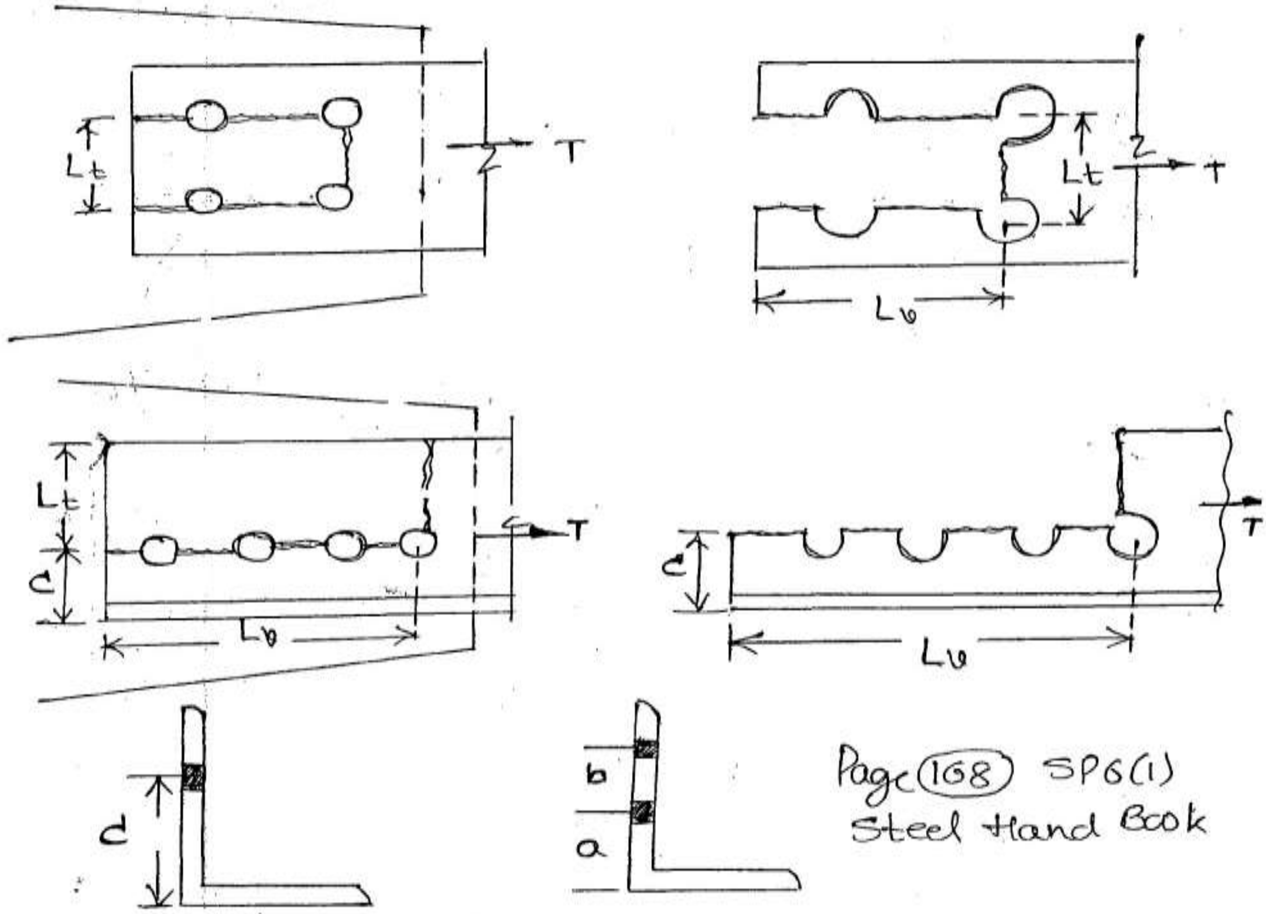
Page (33)

L_c = Distance between outermost bolts.



$L_c = \left(\frac{l_1 + l_2}{2}\right)$ (Take average length)

3. Design strength due to "Block Shear"



Page (168) SP6(1) Steel Hand Book

$$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \frac{0.9\, A_{tn} \cdot f_u}{\gamma_{m1}}$$

Page (33)

&

$$T_{db} = \frac{0.9\, A_{vn} \cdot f_u}{\sqrt{3}\, \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}$$

A_{vg} = Gross area parallel to load = $(L_v \times t)$

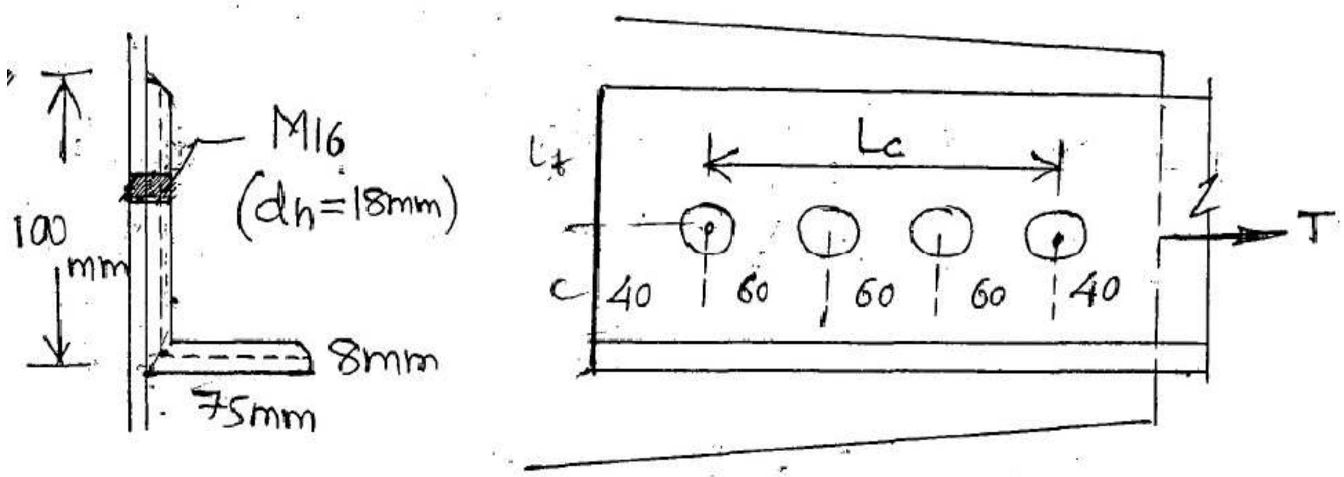
A_{vn} = Net area —"— = $(L_v \times t)$ – Bolt hole.

A_{tg} = Gross area $\perp^r$ to load = $(L_t \times t)$

A_{tn} = Net area —"— = $(L_t \times t)$ – Bolt hole

1. Determine the tensile strength of the tie member ISA 100 mm x 75mm x 8mm connected to gusset plate using 4 bolts of M16 at a pitch of 60mm and edge distance of 40 mm.

Also check for reversal stress or slenderness ratio taking length = 2.5 m.



From steel Table ISA 100 x 75 x 8mm

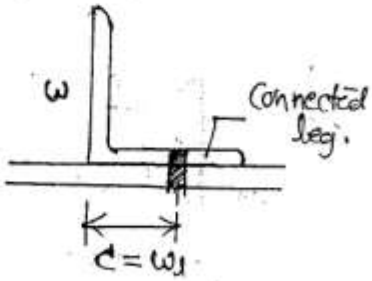
C/s area = 13.36 cm^2 = 1336 mm^2

g = 60mm For leg size 100mm → Page (168) SP(6)

1) Solid or Yield strength

$$\boxed{T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}}} = \frac{1336 \times 250}{1.10} = \boxed{303.63\,kN}$$

(2) Rupture strength of "Angle":



(i) ω = outstanding leg = 75mm

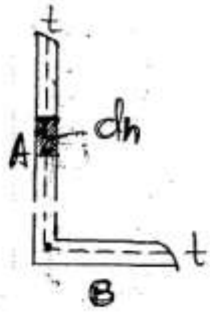
(ii) t = Angle thickness = 8mm

(iii) b_s = shear lag = $(\omega + \omega_1 - t)$

$= (75 + 60 - 8) = 127\text{mm}$

IV) L_c = Dist. betn outer most bolts = 3X pitch = 3X60

$= 180\text{mm}$



V) A_{go} = Gross area of outstanding leg

$= (B - t/2)t = (75 - 8/2) \times 8$

$= 568\text{mm}^2$

A_{nc} = Net area of connected leg

$= (A - d_h - t/2)t = (100 - 18 - 8/2)8 = 624\text{mm}^2$

$$\therefore \beta = 1.4 - 0.076\left(\frac{\omega}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)$$

$$= 1.4 - 0.076\left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{127}{180}\right)$$

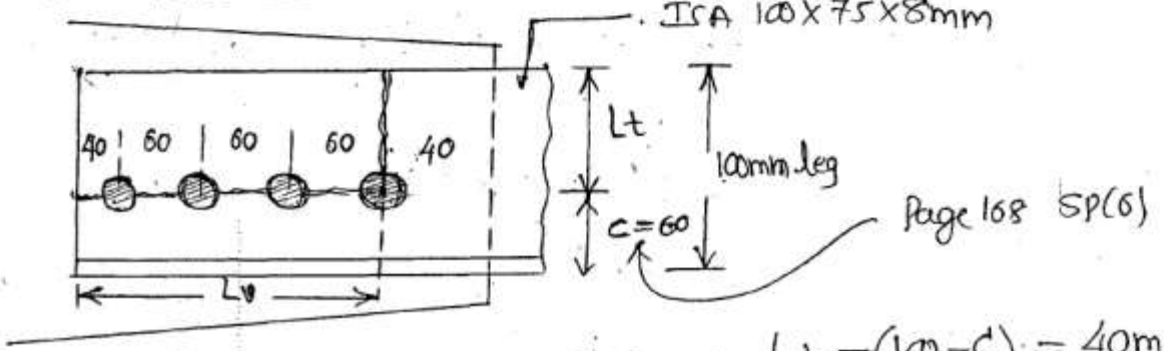
$$= 1.093 \leq \left(\frac{f_u \cdot \gamma_{mo}}{f_y \cdot \gamma_{m1}}\right) = \left(\frac{410 \times 1.10}{250 \times 1.25}\right) = 1.44$$

$\therefore \boxed{\beta = 1.094}$ is In between 0.7 & 1.44

$$\therefore \boxed{T_{dn} = \frac{0.9\, A_{nc} \cdot f_u}{\gamma_{m1}} + \frac{\beta \times A_{go} \cdot f_y}{\gamma_{mo}}}$$

$$= \frac{0.9 \times 624 \times 410}{1.25} + \frac{(1.094)\, 568 \times 250}{1.10} = \boxed{325.4 \text{ kN}}$$

(3) Block shear strength :



$L_v = 3 \times 60 + 40 = 220$ mm, $L_t = (100 - c) = 40$ mm

$\therefore A_{vg} = L_v \times t = 220 \times 8 = 1760 \text{ mm}^2$

$A_{vn} = (1760) - (3.5 d_h \times t)$

$= (1760) - (3.5 \times 18 \times t) = 1256 \text{ mm}^2$

$A_{tg} = L_t \times t = 40 \times 8 = 320 \text{ mm}^2$

$A_{tn} = (320) - (0.5 d_h \times t)$

$= (320) - (0.5 \times 18 \times 8) = 248 \text{ mm}^2$

$$\therefore \boxed{T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} + \frac{0.9\, A_{tn} \cdot f_u}{\gamma_{m1}}}$$

$$= \frac{1760 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 248 \times 410}{1.25} = \boxed{304.15 \text{ kN}}$$

or $\boxed{T_{db} = \frac{0.9\, A_{vn} \cdot f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{mo}}}$

$$T_{db} = \frac{0.9 \times 1256 \times 410}{\sqrt{3} \times 1.25} + \frac{320 \times 250}{1.10} = \boxed{286.8\ kN}$$

$\therefore$ Tensile strength of ISA 100X75X8mm $\Big\}$ = Least of T_{dg}, T_{dn}, T_{db}

$= 286.8\ kN$

(4) Check for "Slenderness Ratio":

From steel Table ISA 100X75X8mm

$\gamma_x = 3.14$ cm
$\gamma_y = 2.18$ cm
$\gamma_u = 3.48$ cm
$\gamma_v = 1.59$ cm

$\Big\}$ $\gamma_{min} = 1.59\text{cm} = 15.9\text{mm}$

Take effective length $\boxed{l_e = 0.85 l} = 0.85 \times 2.5\text{m}$

$= 2.125\text{m}$

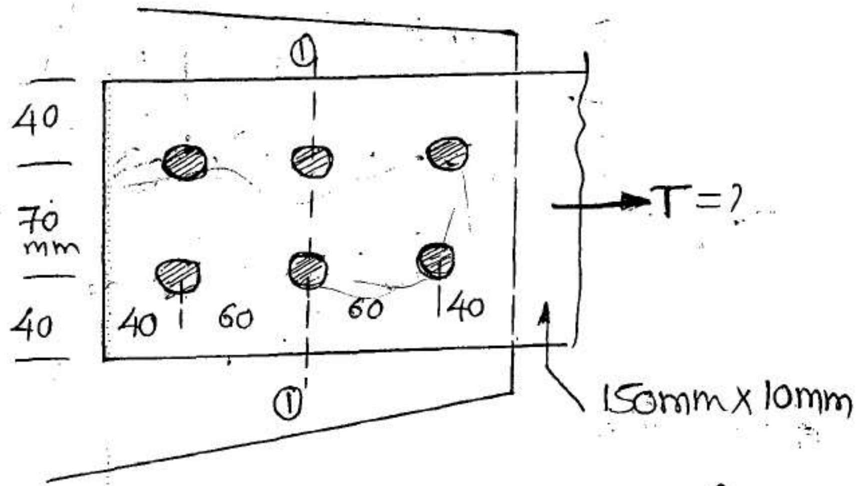
$$\therefore \boxed{\lambda = \frac{l_e}{\gamma_{min}}} = \frac{2125\text{mm}}{15.9\text{mm}} = \boxed{133.64} < 180$$

< 350

(safe)

2. Determine the tensile strength of a plate 150 mm x 10 mm connected to gusset plate using 6 bolts of M18. Take pitch = 60mm and e = 40 mm.

3)



(a) "Yield" strength of Plate : (Page 32)

$$\boxed{T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}}} = \frac{(150\times10)(250)}{1.10} = \boxed{340.9\,kN}$$

(b) "Rupture" strength of Plate : (Page 32)

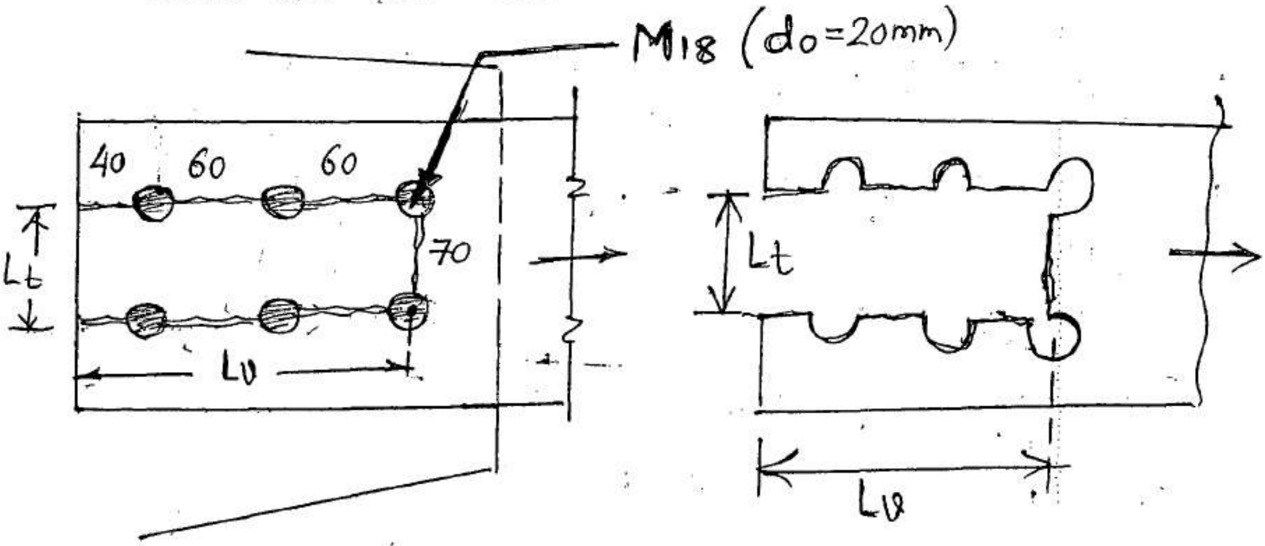
$$\boxed{T_{dn} = \frac{0.9\,A_n \cdot f_u}{\gamma_{m1}}}$$

$$A_n = \left[b - n \cdot d_h + \cancel{\sum \frac{P_s^2}{4g}}\right] t$$

$$= \left[150 - 2\times20\right]\times10 = 1100\,mm^2$$

$$T_{dn} = \frac{0.9 \times 1100 \times 410}{1.25} = \boxed{324.72\,kN}$$

(c) Block shear strength (Page 33)



$L_v = 40 + 2\times 60 = 160\,mm$

$L_t = 70\,mm$

$\therefore A_{vg} = (L_v \times t)2^{*}$

For Two line parallel to force

$= (160\times 10)2 = 3200\,mm^2$

$A_{tg} = (L_t \times t) = (70\times 10) = 700\,mm^2$

$A_{vn} = 3200 - \overset{*}{2}(2.5 \times d_o \times t)$

$= 3200 - 2(2.5\times 20\times 10) = 2200\,mm^2$

$A_{tn} = 700 - 2(0.5 d_o \times t) = 700 - 2(0.5\times 20\times 10)$

$= 500\,mm^2$

$$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{mo}} + \frac{0.9\, A_{tn} \cdot f_u}{\gamma_{m1}}$$

$$= \frac{3200 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 500 \times 410}{1.25} = \boxed{567.50\ kN}$$

or $$T_{db} = \frac{0.9\, A_{vn} \cdot f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{mo}}$$

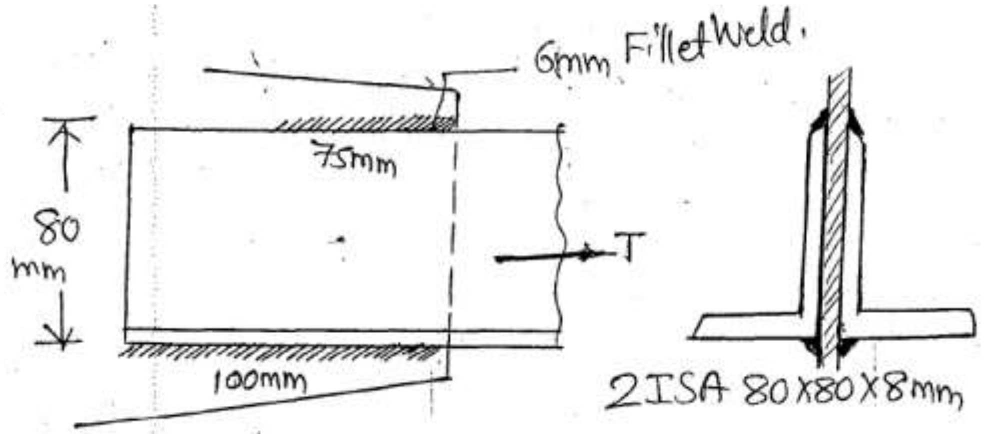
$$= \frac{0.9 \times 2200 \times 410}{\sqrt{3} \times 1.25} + \frac{700 \times 250}{1.10} = \boxed{534.0\ kN}$$

∴ Tensile Strength of Plate } = Least of T_{dg}, T_{dn}, T_{db}

= 324.72 kN

3. Determine tensile strength of a Tie member 2 ISA 80 x 80 x 8mm connected to gusset plate on either side using 6mm fillet weld. The length of the weld is 100mm and 75mm.

Sol



(a) Yield strength :

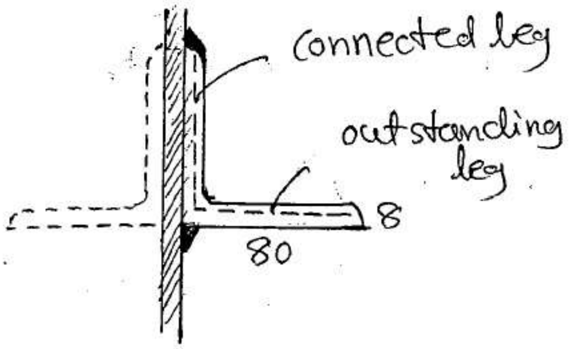
$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{2442 \times 250}{1.10}$$

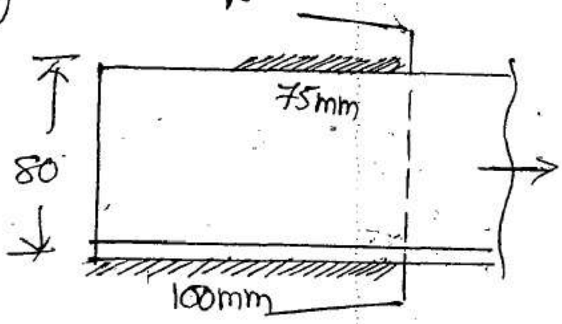
$$= \boxed{555.0\,kN}$$

C/s area of 2 ISA 80x80x8mm $= 24.42\,cm^2$ $= 2442\,mm^2$

(b) Rupture strength of "Angle" (Page 33)

★ Do the calculation for one angle and at the end multiply by "2". ★





$\omega \rightarrow$ outstanding leg = 80mm

$t \rightarrow$ Angle thickness = 8mm

f_y & f_u $\rightarrow$ 250 & 410

$b_s = \omega = 80mm$

★

L_c = Average length of Weld $= \left(\frac{100+75}{2}\right) = 87.5mm$

$$\beta = 1.4 - 0.076\left(\frac{\omega}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L}\right) \leq \left(\frac{f_u \cdot \gamma_{m0}}{f_y \cdot \gamma_{m1}}\right)$$

$$\geq 0.7$$

$$\beta = 1.4 - 0.076\left(\frac{80}{8}\right)\left(\frac{250}{410}\right)\left(\frac{80}{87.5}\right) \leq \left(\frac{410\times1.10}{250\times1.25}\right)$$

$$\therefore \boxed{\beta = 0.97} \qquad = 1.44$$

$$\geq 0.7$$

Hence β is inbetween 0.7 & 1.44.

$$\therefore T_{dn} = \frac{0.9 A_{nc} \cdot f_u}{\gamma_{m_1}} + \frac{\beta A_{go} \cdot f_y}{\gamma_{mo}}$$

A_{go} = Gross area of outstanding leg $= \left(B - \frac{t}{2}\right)t$

$$= \left(80 - \frac{8}{2}\right)8 = 608\,mm^2$$

A_{nc} = Net area of connected leg $= \left(A - \cancel{d_0} - \frac{t}{2}\right)t$

$$= \left(80 - 8/2\right)8 = 608\,mm^2$$

$$\therefore T_{dn} = \frac{0.9\times608\times410}{1.25} + \frac{(0.97)608\times250}{1.10} = \boxed{313.5\,kN}$$

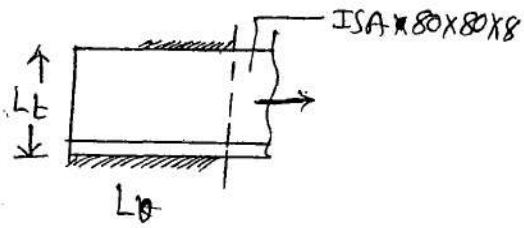
$\therefore$ Total $= 2\times313.5 = 627\,kN$

Two angle

(e) Block Shear Failure :

L_v = Average length of weld $= \frac{100+75}{2} = 87.5\,mm$

$L_t = 80$



$\therefore A_{vg} = A_{vn} = L_v \times t = 87.5\times8$

$$= 700\,mm^2$$

$A_{tg} = A_{tn} = L_t \times t = 80\times8 = 640\,mm^2$

$$T_{db} = \left[\frac{700 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 640 \times 410}{1.25}\right] \times 2 = \boxed{561.55\,kN}$$

$$\& \; T_{db} = \left[\frac{0.9 \times 700 \times 410}{\sqrt{3} \times 1.25} + \frac{640 \times 250}{1.10}\right] \times 2 = \boxed{529.52\,kN}$$

$\therefore$ Tensile strength of 2 ISA 80X80X8 $\Big\} =$ 529.52 kN

Design of Tension Members

Following data's are given:

Force, Type of Section, Type of Connection

Following steps are used in the design of tension member

1. Calculate Gross area required using the formula

$$Ag = \frac{Factored\ load\ \ X\ \gamma_{mo}}{fy}$$

Increase the above area by 30% approximately

From the steel table select suitable section

2. Calculate the bolt value

3. Calculate the number of bolts

$$\text{No. of bolts} = \frac{Factored\ load}{Bolt\ value}$$

4. Draw the neat sketch showing the arrangement of bolts.

5. Calculate Tdg, Tdn and Tdb (Page 32 and 33)

If Tdn, Tdn and Tdb > Given force, then safe otherwise revise the section.

6. Check for slenderness Ratio.

1. Design a tie member consisting of single angle section to carry a working load of 150 KN. Use bolted connection with M18 property class 5.6 bolts.

   If the length of the member is 2m , check for slenderness ratio.

Sol:

Ultimate or Factored load $= 1.5 \times 150 = \boxed{225\,kN}$

↑ P.S.F

(a) Gross area Required $= \dfrac{(\text{Factored load})\,\gamma_{mo}}{f_y}$

$$A_g = \frac{(225 \times 10^3)\,1.10}{250} = 990\,mm^2$$

Increase by 30% approximately $= 1.30 \times 990$

$= 1287\,mm^2 = \underline{12.87\,cm^2}$

From steel Table Try $\underline{ISA\ 100 \times 75 \times 8\,mm}$

(Area $= 13.36\,cm^2$).

(b) Connections :-

M18 → Property class 5.6

Pitch $= 2.5 \times \overset{d}{18} = \boxed{45\,mm}$

Edge distance $= 1.7 \times d_0 = 1.7 \times 20 = 34\,mm \approx \boxed{35\,mm}$

Bolt Value :

In shear — Assume Thread in shear plane

$\therefore n_n = 1, \quad n_s = 0, \quad A_{nb} = 0.78 \times \frac{\pi}{4}(18)^2 = 198.48$

$\therefore V_{nsb} = \frac{410}{\sqrt{3}}(1 \times 198.48 + 0) = 46.98 \text{ kN}$

$\therefore$ Design shear strength $= V_{dsb} = \frac{46.98}{1.25} = \boxed{37.58 \text{ kN}}$

In bearing

$k_b \longrightarrow$ (i) $\frac{e}{3d_0} = \frac{35}{3 \times 20} = 0.58$

(ii) $\left(\frac{p}{3d_0} - 0.25\right) = \left(\frac{45}{3 \times 20} - 0.25\right) = 0.5$

(iii) $\frac{500}{410} = 1.21$, (iv) 1.0

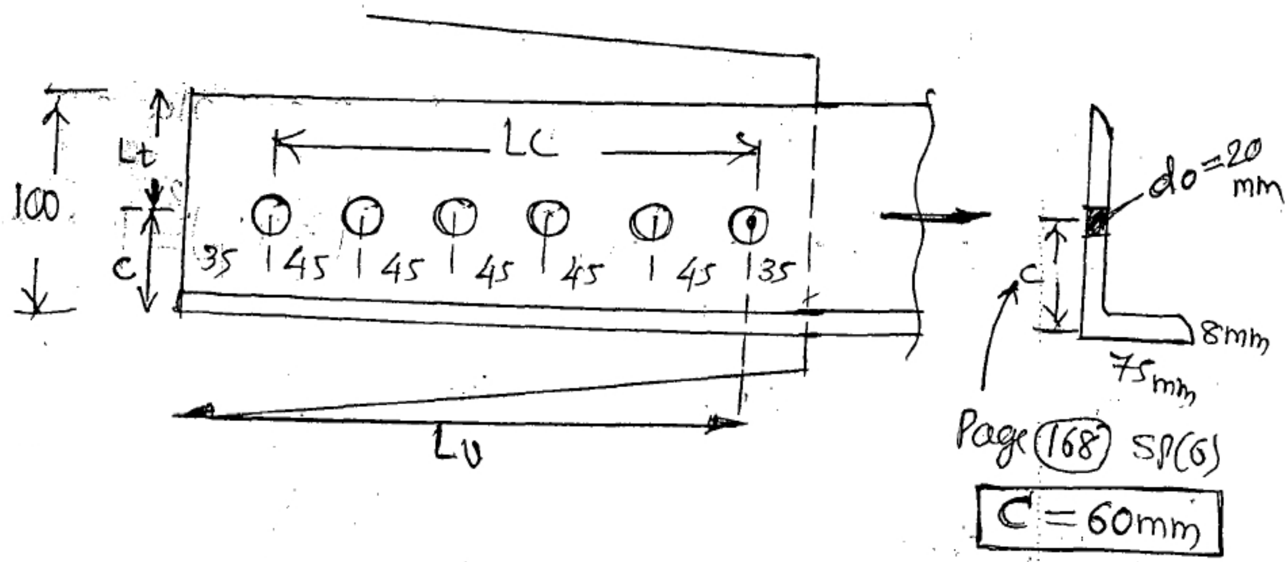
$\therefore \boxed{k_b = 0.5}$

$\therefore V_{npb} = 2.5 \times 0.5 \times 18 \times 8 \times 410 = 73.8 \text{ kN}$

$\therefore V_{dpb} = \frac{73.8}{1.25} = \boxed{59.04 \text{ kN}}$

$\therefore \boxed{\text{Bolt Value} = 37.58 \text{ kN}}$

$\therefore$ No. of Bolts $= \frac{\text{Force}}{\text{B.V}} = \frac{225}{37.58} \approx 6$



(c) Yield strength :

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{mo}} = \frac{(1336)\,250}{1.10} = 303.6\,kN > 225\,kN \text{ (safe)}$$

(d) Rupture Strength :

$w = 75mm$, $t = 8mm$

$b_s = (w + w_1 - t) = (75 + 60 - 8) = 127mm$

$L_c = 5 \times p = 5 \times 45 = 225mm$

$$\therefore \beta = 1.4 - 0.076\left(\frac{75}{8}\right)\left(\frac{250}{410}\right)\left(\frac{127}{225}\right)$$

$\boxed{\beta = 1.157}$ > 0.7
< 1.44

$A_{go} = \left(B - \frac{t}{2}\right)t = \left(75 - \frac{8}{2}\right)8 = 568$

$A_{nc} = \left(A - d_o - \frac{t}{2}\right)t = \left(100 - 20 - \frac{8}{2}\right)8 = 608$

$$T_{dn} = \frac{0.9 \times 608 \times 410}{1.25} + \frac{(1.157)\, 568 \times 250}{1.10}$$

$$T_{dn} = 328.8\,kN > 225\,kN \text{ (Safe)}$$

(e) Block Shear :

$L_v = 35 + 5 \times 45 = 260\,mm$

$L_t = (100 - c) = (100 - 60) = 40\,mm$

$A_{vg} = L_v \times t = 260 \times 8 = 2080$

$A_{tg} = L_t \times t = 40 \times 8 = 320$

$A_{vn} = 2080 - (5.5 \times \underset{d_o}{20} \times \underset{t}{8}) = 1200$

$A_{tn} = 320 - (0.5 \times 20 \times 8) = 240$

$$T_{db} = \frac{2080 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 240 \times 410}{1.25} = 343.7\,kN > 225\,kN \text{ (Safe)}$$

&

$$T_{db} = \frac{0.9 \times 1200 \times 410}{\sqrt{3} \times 1.25} + \frac{320 \times 250}{1.10} = 277.2\,kN > 225\,kN \text{ (Safe)}$$

Hence Provide

ISA 100x75x 8mm with "6" Bolts

of M18 Property class – 5.6.

(f) Check for slenderness Ratio

$\boxed{l_e = 0.85 l} = 0.85 \times 2 = 1.70\,m$

From steel Table ISA $100 \times \cancel{80}\,75 \times 8\,mm$

$r_{xx} = 3.14\,cm$

$r_{yy} = 2.18\,cm$

$r_{uu} = 3.48\,cm$

$r_{vv} = 1.59\,cm$

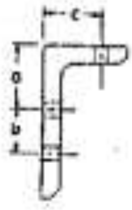
$\therefore r_{min} = 15.9\,mm$

$\therefore \lambda = \frac{l_e}{r_{min}} = \frac{1700}{15.9} = 108.9 < 180$

< 350

(Safe)

TABLE XXXI RIVET GAUGE DISTANCES IN LEGS OF ANGLES



Leg Size	Double Row of Rivets		Single Row of Rivets	Maximum Rivet Size for Double Row of Rivets
	a	b	c	
mm	mm	mm	mm	mm
200	75	85	115	27
150	55	65	90	22
130	50	55	80	20
125	45	55	75	20
115	45	50	70	12
110	45	45	65	12
100	40	40	60	12
95	—	—	55	—
90	—	—	50	—
80	—	—	45	—
75	—	—	40	—
70	—	—	40	—
65	—	—	35	—
60	—	—	35	—
55	—	—	30	—
50	—	—	28	—
45	—	—	25	—
40	—	—	21	—
35	—	—	19	—
30	—	—	17	—
25	—	—	15	—
20	—	—	12	—

3. Design a tie member to carry an axial load of 400 KN (Working). Use Double angles with M20 HSFG Bolts and property Class 10.9.

Sol^n

Ultimate load = 1.5 × 400 = 600 kN

(a) Area Required :

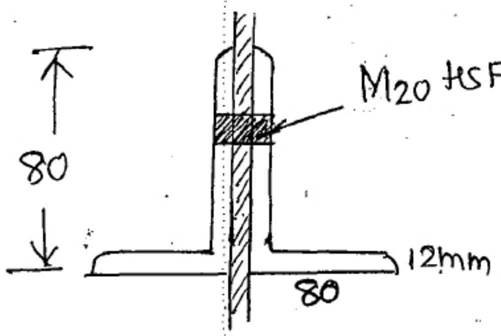
$$A_g = \frac{600 \times 10^3 \times 1.10}{250} = 2640 mm^2 = 26.4 cm^2$$

(γ_{mo} = 1.10, f_y = 250)

Increase by 30% = 1.30 × 26.4 = 34.32 cm²

From steel Table 2 ISA 80×80×12mm

(Area = 35.62 cm²).



(b) Bolted Connection : [Non Slip Joint]

Given M20 HSFG Bolts Property class 10.9

Page (76)

μ_f = Slip factor = 0.55

n_e = No. of Interface = 2

K_h = 1.0 (For clearance hole).

$$\therefore F_0 = 245.04 \times 0.7 \times 1000 = 171.53 \times 10^3 \, N$$

$$\therefore \text{Nominal Shear Capacity} \Big\} = \boxed{V_{nsf} = \mu_f \times n_e \times K_h \times F_0}$$

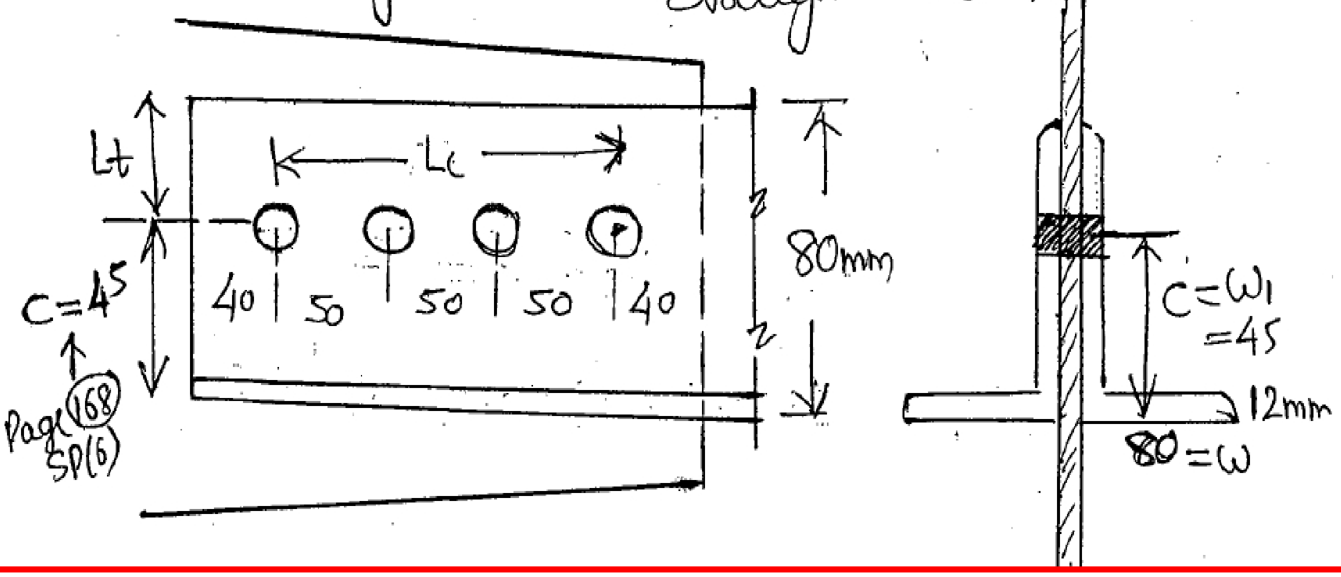
Page (76)

$$V_{nsf} = (0.55)(2)(1)(171.53 \times 10^3) = 188.68 \, kN$$

$$\therefore \text{Design Value} = \boxed{V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}}}$$

$$V_{dsf} = \frac{188.68}{1.25} = \boxed{150.94 \, kN}$$

$$\therefore \text{No. of Bolts} = \frac{\text{Force}}{\text{Strength}} = \frac{600}{150.94} \approx 4$$



(C) Check for the strength

(i) Yield strength :

$$T_{dg} = \frac{(3562) \times 250}{1.10} = 809.5 \text{ kN} > 600 \text{ kN (Safe)}.$$

(ii) Rupture strength :

$$\beta = 1.4 - 0.076\left(\frac{80}{12}\right)\left(\frac{250}{410}\right)\left(\frac{\overset{w}{80} + \overset{w_1}{45} - \overset{t}{12}}{150}\right)$$

$$\boxed{\beta = 1.17} \begin{array}{l} > 0.7 \\ < 1.44 \end{array} \text{ (Ok)}$$

$$A_{go} = \left(B - \frac{t}{2}\right)t = \left(80 - \frac{12}{2}\right) \times 12 = 888$$

$$A_{nc} = \left(A - d_0 - \frac{t}{2}\right)t = \left(80 - 22 - \frac{12}{2}\right)12 = 624$$

$$\therefore T_{dn} = \left[\frac{0.9 \times 624 \times 410}{1.25} + \frac{(1.17) 888 \times 250}{1.10}\right] 2$$ ← For double angle

$$\therefore T_{dn} = 840.6 \text{ kN} > 600 \text{ kN (Safe)}.$$

(iii) Block Shear Strength :

$$L_v = 40 + 3 \times 50 = 190, \quad L_t = (80 - 45) = 35 \text{ mm}$$

$$A_{vg} = L_v \times t = 190 \times 12 = 2280$$

$$A_{vn} = 2280 - (3.5 \times 22 \times 12) = 1356$$

$$A_{tg} = L_t \times t = 35 \times 12 = 420$$

$$A_{tn} = 420 - (0.5 \times 22 \times 12) = 288$$

∴ $T_{db} = 768.38\ kN > 600\ kN$ } Safe

or $T_{db} = 653.12\ kN > 600\ kN$

∴ Hence Provide 2 ISA 80X80X12mm

with 4 Bolts of M20 HSFG. Property class 10.9.

4. Design a Tie member which consists of single angle section to carry a tensile force of 200 KN. The length of the member is 3.5 m and subjected to reversal of stress due to wind force.
   The yield strength and ultimate strength of steel used are 250 Mpa and 410 Mpa and using 20 mm bolts.

Given Data:

Service load = 200 KN

Factored load = 1.5 x 200 = 300 KN

Length = 3.5 m

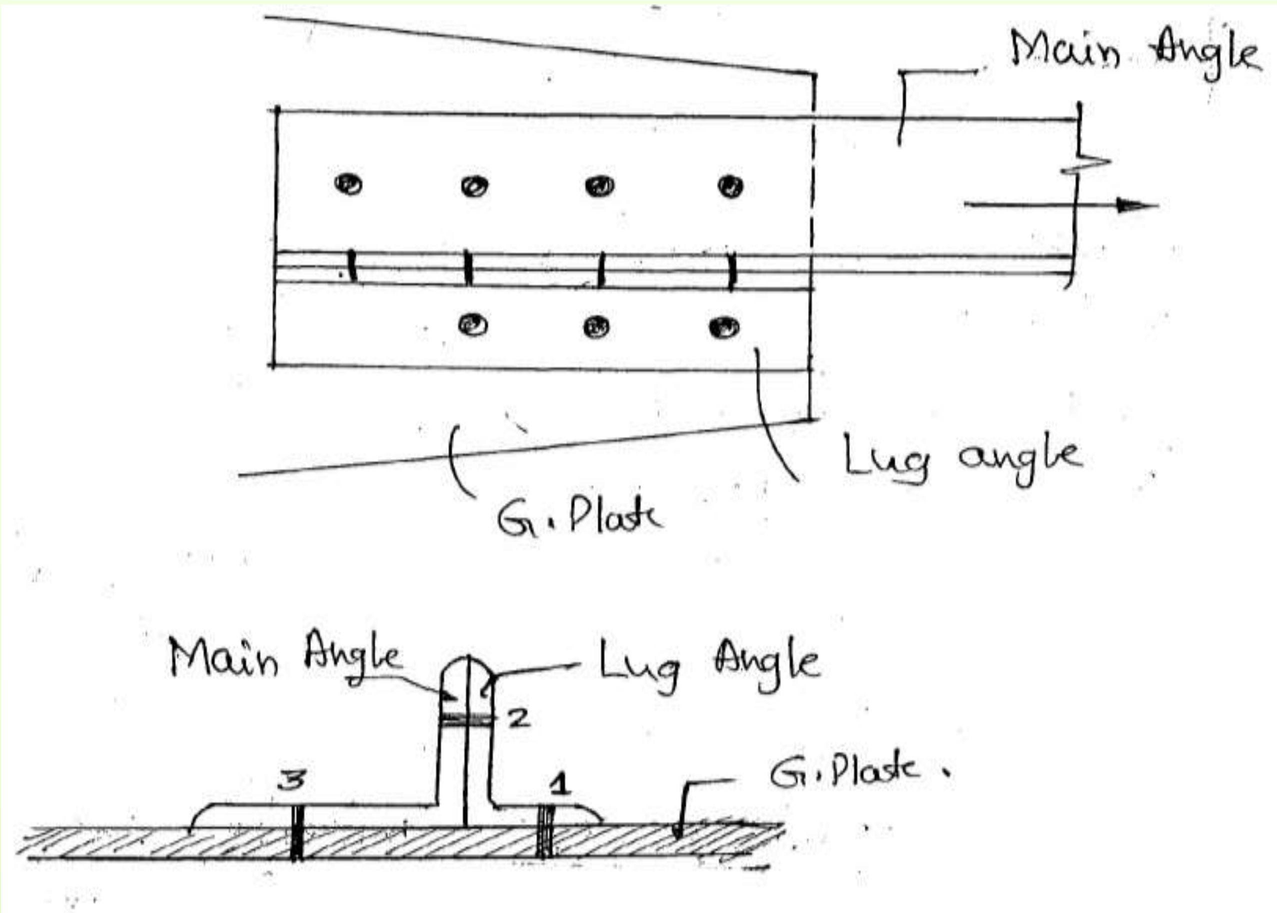
Fy = 250 Mpa

Fy = 410 N/mm

D = 20 mm

Do = 20 + 2 = 22 mm

LUG ANGLES :



- ✓ Lug angles are short angles used to connect the gusset plate and the outstanding leg of the main member as shown in figure.
- ✓ The lug angle helps to increase the efficiency of the outstanding leg of angle.
- ✓ They are normally provided when the tension member carries a very large load.
- ✓ Higher load results in a larger end connection which can be reduced by providing lug angles.

- ✓ It is ideal to place the lug angle at the beginning of the connection then at any position.
- ✓ If the length of the connection is more which can be reduced by lug angle.

Advantages:

The only advantage is length of the connection can be reduced.

Disadvantages:

- ✓ The connection requires additional piece of angle section.
- ✓ The overall number of bolts is more than without the lug angle connection
- ✓ The lug angle connection is slightly eccentric.

Design specifications of Lug angles:

a. For the design of lug angles, increase the force in outstanding leg by 20% (for channel section 10%)

b. For connection between lug angle and gusset plate the force in outstanding leg is increased by 30% (for channel section 10%)

c. For connection between lug angle and main angle the force in the outstanding leg is increased by 40% (for channel section)

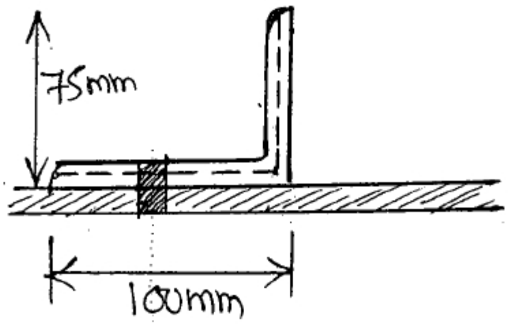
Design of Tension Member with Lug Angles:

Following procedure is adopted for the design of tension member with lug angle

1. Design of Tension Member
2. Design of Lug angle.
3. Connections
    a. Find Bolt value which is least of Shear and Bearing strength.
    b. Connection between lug angle and Gusset plate.
    c. Connection between lug angle and main angle.
    d. Connection between main angle and Gusset plate.

1. The tie member ISA 100 x 75 x 8 mm carries a load of 300 KN (factored). Design a lug angle connection using M18 property class 5.6 bolts.

Tie Member → ISA 100 X 75 X 8 mm



(a) Gross area of connected leg $= \left(100 - \frac{8}{2}\right)8 = 768\,mm^2$

Gross area of outstanding leg $= \left(75 - \frac{8}{2}\right)8 = 568\,mm^2$

∴ Load carried by "connected leg" $= \frac{768}{(768+568)} \times 300 = \boxed{172.45\,kN}$

& load carried by "outstanding leg" $= (300) - (172.45)$

$= \boxed{127.55\,kN}$

(b) Design Lug Angle :

* According to code → Increase the force in outstanding leg by 20 % $= 1.20 \times 127.55$

$= 153.06\,kN$

$$\therefore \text{Area})_{Req} = \frac{(\text{Load})\,\gamma_{mo}}{f_y} = \frac{(153.06 \times 10^3)\,1.10}{250}$$

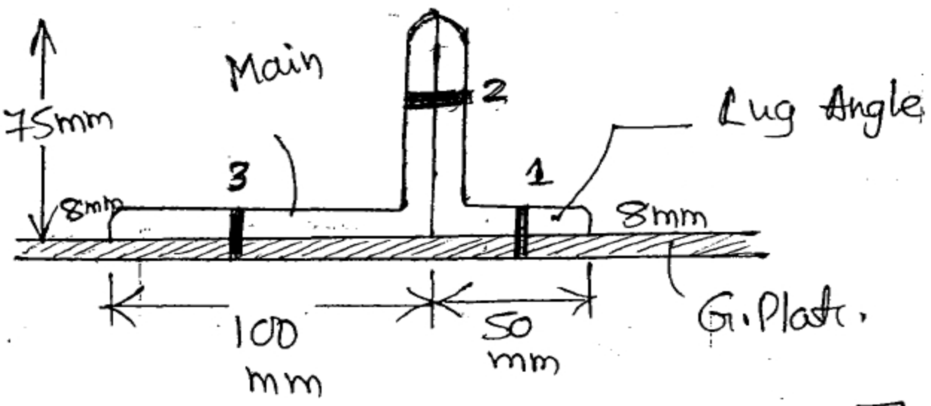
$$= 673.46\ mm^2$$

Increase by 30% approximately $= 1.30 \times 673.46$

$= 875.50\ mm^2$

$= 8.75\ cm^2$

Provide ISA 75x50x8mm ($9.38\ cm^2$)



(c) Connections : [1, 2 & 3]

(i). In shear : Assume 'Threads' in shear plane $n_n = 1$, $n_s = 0$

$$\therefore A_{nb} = 0.78 \times \frac{\pi}{4}(18)^2 = 198.48\ mm^2$$

$$\therefore V_{nsb} = \frac{500}{\sqrt{3}}(1 \times 198.48 + 0) = 57.29\ kN$$

$$\therefore V_{dsb} = \frac{57.29}{1.25} = 45.83\ kN$$

(ii) In bearing :-

$e = 1.7 \times d_0 = 1.7 \times 20 = 35mm$

$p = 2.5 \times d = 2.5 \times 18 = 45mm$

$k_b \rightarrow$ (1) $\frac{e}{3d_0} = \frac{35}{3 \times 20} = 0.58$ | (3) $\frac{500}{410} = 1.21$

(2) $\left(\frac{p}{3d_0} - 0.25\right) = 0.50$ | (4) 1.0

$\therefore k_b = 0.50$

$V_{npb} = 2.5 \times 0.5 \times 18 \times 8 \times 410 = 73.8 kN$

$\therefore$ design $V_{dpb} = \frac{73.8}{1.25} = 59.04 kN$

$\therefore$ Least $\rightarrow$ $\boxed{\text{Bolt Value} = 45.83 kN}$

(1) Connection betn Lug angle & G. Plate :

* According to code $\rightarrow$ The force in the outstanding leg is increased by 20% $= 1.20 \times 127.55$
$= 153.06 kN$

$\therefore$ No. of Bolts $= \frac{\text{Force}}{\text{B.V}} = \frac{153.06}{45.83} \approx 4$

② Connection betⁿ Lug angle & Main Angle:

* The force is increased by 40%

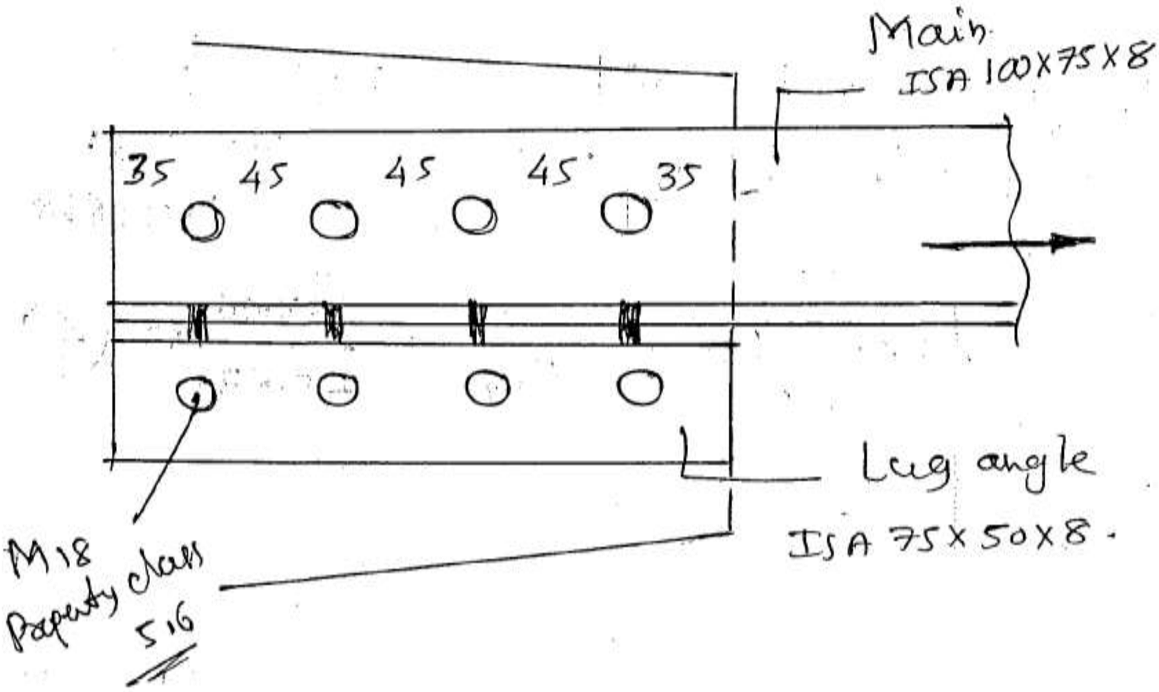
$$= 1.40 \times 127.55 = 178.57 \text{ kN}$$

$$\therefore \text{No. of Bolts} = \frac{178.57}{45.83} \approx 4$$

③ Connection betⁿ Main & G.P :-

$$\left.\begin{array}{l}\text{Load carried by} \\ \text{connected leg}\end{array}\right\} = 172.4 \text{ kN}$$

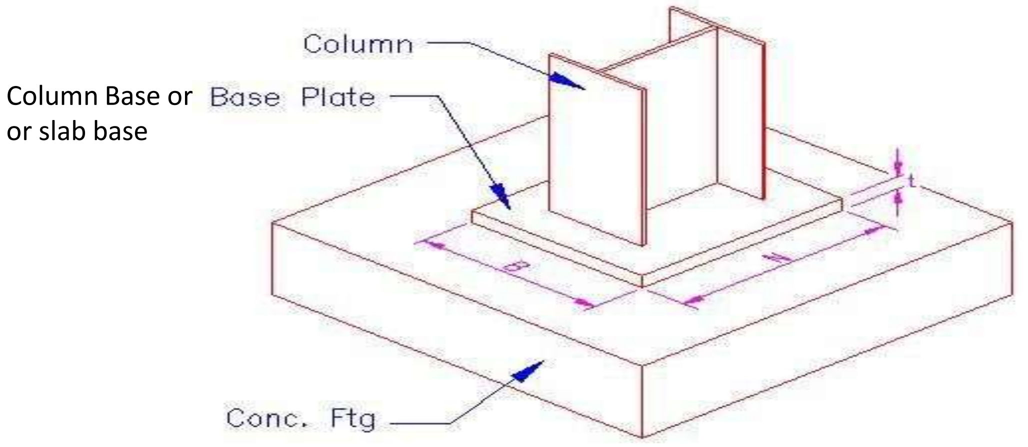
$$\therefore \text{No. of Bolts} = \frac{172.4}{45.83} \approx 4$$



DESIGN OF STEEL STRUCTURAL ELEMENTS
(18CV61)

MODULE 4
DESIGN OF COLUMN BASES

Column Bases:



- ✓ The columns are supported on the column base.
- ✓ The column base is provided for transferring the load from the column to the base and to distribute it evenly on the concrete bed.
- ✓ The load is also distributed over a larger area, so that the stress induced in the concrete is within its permissible limits and is capable of resisting overturning.
- ✓ If column base is not provided, the column is likely to punch through the concrete block.
- ✓ Mild steel plates of sufficient area are attached to the bottom of the column in order to increase the bearing area. Such plates are called column bases.
- ✓ These plates are secured to the concrete block through holding down or anchor bolts.

There are two types of column bases:

1) Slab base
2) Gusseted base.

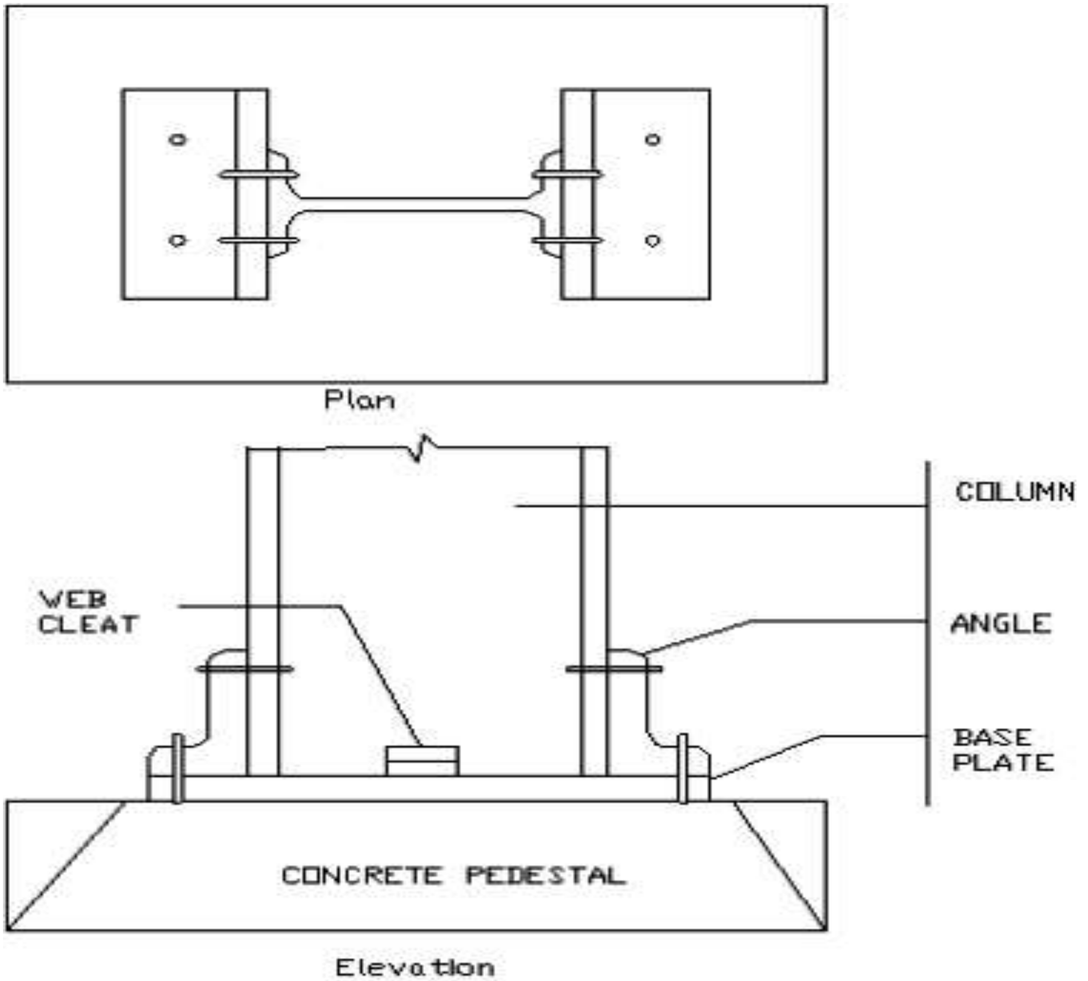
Purpose of providing a column base.

Column bases are provided for following purposes

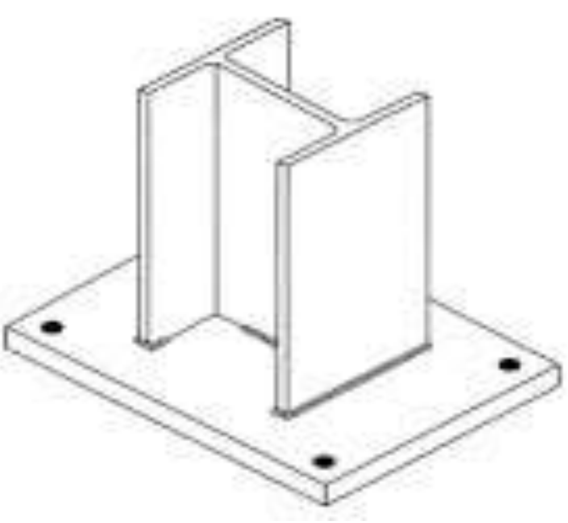
a. to distribute the column load to concrete pedestal or blocks
b. to maintain alignment of column in plan
c. to maintain verticality of column
d. to control deflection of column and frames.

Slab Base:

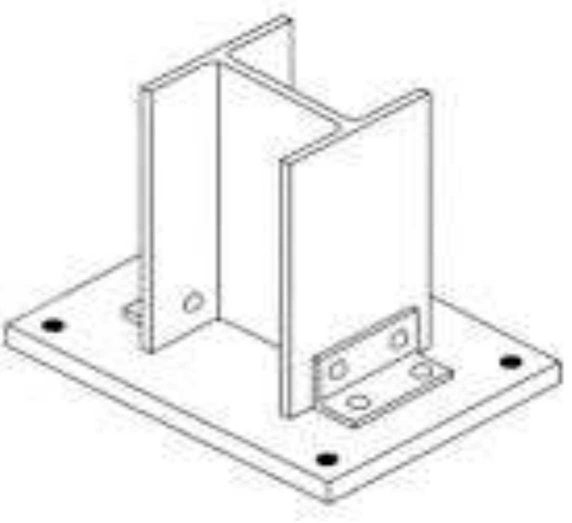
- For columns carrying small loads, slab bases are used.
- It consists of a base plate, placed below the machined column end and cleat angles.
- The machined column end transfers the load to the slab base by direct bearing.
- The column end is connected to base plate by welding or by means of bolts.
- In order to prevent movement of plate in the horizontal plane, four bolts are provided in the four corners of the plate and this bolts are called as anchor bolts or holding down bolts.



PLAN AND ELEVATION OF SLAB BASE



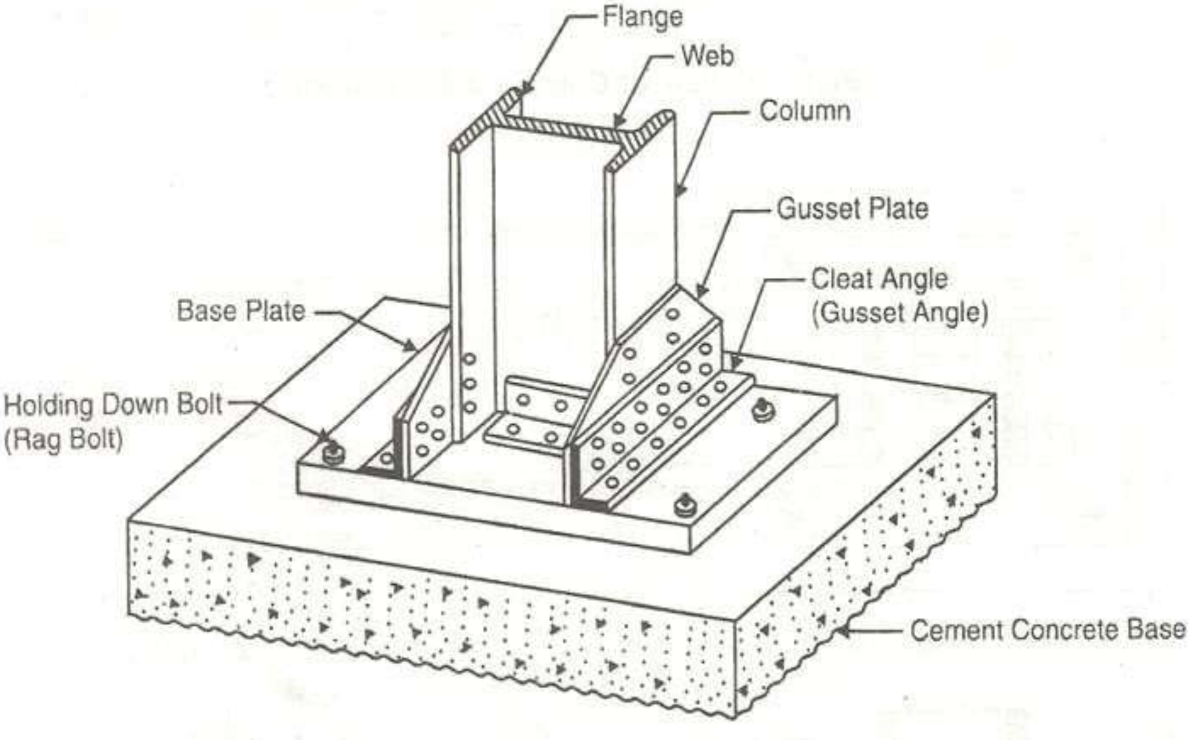
Slab base with Welded Connection

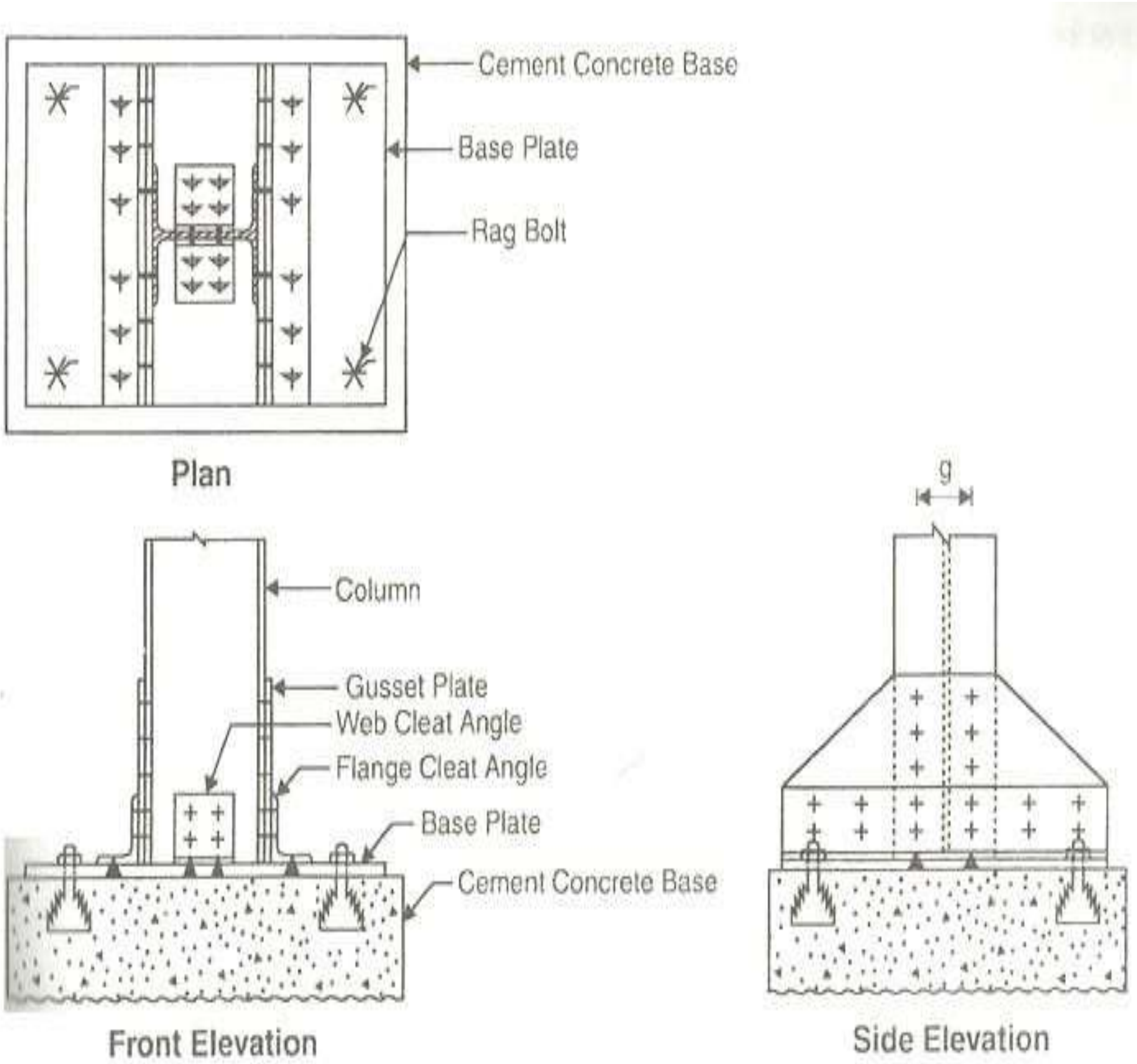


Slab base with Bolted Connection

Gusseted Base:

- ✓ For columns carrying heavy loads gusseted bases are used.
- ✓ The loads are transmitted to the base plate through the gusset plates attached to the flanges of the column by means of cleat angles.
- ✓ So the gusseted base consists of base plate, gusset plates and cleat angles or gusset angels.
- ✓ The base plate is anchored at the four corners to the foundation with bolts to check the lateral movement.
- ✓ The foundation is generally of cement concrete and transmits the load over a larger area with uniform distribution of pressure.





Plan, Front Elevation and Side Elevation of Gusseted Base.

Following steps are used to design a slab base:

i. Area of the slab base (base plate)

$$Area = \frac{Factored\ load}{Bearing\ strength\ of\ Concree}$$

- Bearing strength of concrete = 0.45 x fck
- Find the projections ‘a’ and ‘b ‘ by using the equation
- Area = (h+2a) * (bf*2b)
- For Economy, as far as possible take the values of a & b as same.

ii. Thickness of base slab:

Thickness of base slab is calculated by using the equation:

$$t_s = \sqrt{\frac{2.5w(a^2-0.3b^2)\,\gamma_{mo}}{f_y}} > t_f \text{ ....page 47}$$

Where w = Uniform upward pressure from concrete

= Load/Area of base plate

a & b = Larger & smaller projection respectively of the slab beyond the rectangle circumscribing the column

t_f = Flange thickness of compression member.

iii. Connection:

a. Welded connection

Assuming size of the weld s = 8mm and equating

Force = Strength of the weld, Find the length of the weld

i.e., Force = 0.7 x s x L x $\frac{fu}{\sqrt{3}\gamma_{mw}}$

b. Bolted connection

$$\text{No. of Bolts} = \frac{Factored\ Load}{Bolt\ Value}$$

iv. Design of Concrete Base:

$$\text{Find the area of Concrete base} = \frac{Total\ Load}{SBC\ of\ soil}$$

Problems:

1. Design a slab base for a column ISHB 300 @ 583.8 Kg/m subjected to a service load of 1500 KN. The grade of concrete for pedestal is M20 and SBC of soil is 180 KN/m². Design slab base and concrete base with welded connection.

(a) Size of Slab Base :

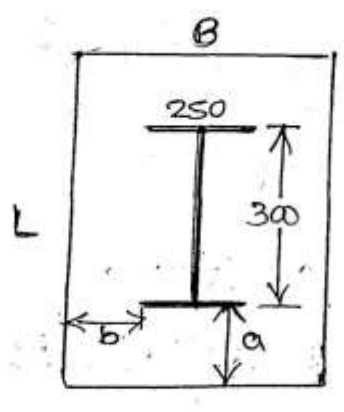
Column load = 1500 kN

$\therefore$ Ultimate load = $1.5 \times 1500 = 2250$ kN

$$\boxed{\text{Bearing Capacity of Concrete} = 0.45 f_{ck}}$$

$\therefore$ Area of slab Base = $L \times B = \dfrac{\text{load}}{0.45 f_{ck}}$

$$L \times B = \frac{2250 \times 10^3 N}{0.45 \times 20} = 250 \times 10^3 \text{ mm}^2 \longrightarrow (i)$$



$\therefore L = (300 + 2a)$

$B = (250 + 2b)$

Provide same projection

(a = b)

$\therefore$ From(i)

$$(300+2a)(250+2a) = 250\times10^3$$

$$\therefore \quad a = 112.8\,mm$$

Take $\boxed{a = b = 120\,mm}$

$$\therefore \quad L = (300+2\times120) = 540\,mm$$

$$B = (250+2\times120) = 490\,mm$$

(b) Thickness of "Slab Base" : (Page 47)

$$t_s = \sqrt{2.5\,\omega\,(a^2 - 0.3b^2)\frac{\gamma_{mo}}{f_y}} > t_f$$

ω = upward soil pressure or concrete pressure

$$\omega = \frac{2250\times10^3}{L\times B} = \frac{2250\times10^3}{540\times490} = 8.50\,N/mm^2$$

$$\therefore t_s = \sqrt{2.5\times8.5\,(120^2 - 0.3\times120^2)\times\frac{1.10}{250}}$$

$$t_s = 30.70\,mm > t_{f\,(=10.6mm)} \quad (Ok)$$

Provide $\boxed{\text{Slab base } 540mm\times490mm\times32mm}$

(c) Design of Concrete Base :

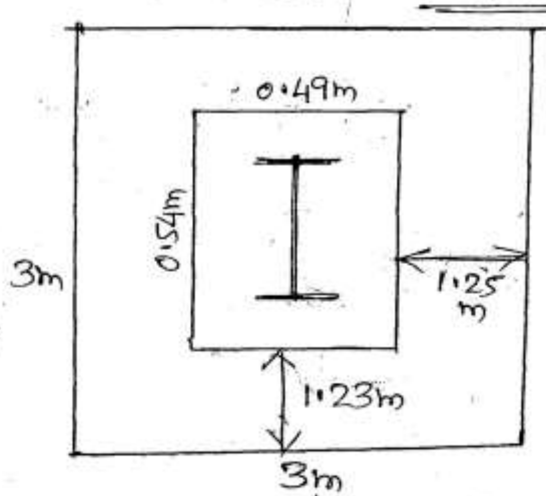
Load on column = 1500 kN
Self wt. of concrete = 150 kN (10% of col. load)
1650 kN Working load.

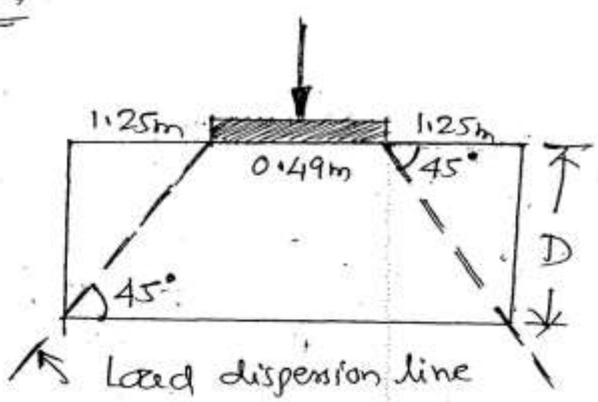
$\therefore$ Area of concrete Base $= \dfrac{1650}{\text{SBC of soil}} = \dfrac{1650 \text{ kN}}{180 \text{ kN/m}^2}$

$= 9.16 \text{ m}^2$

Provide square base $= \sqrt{9.16} = 3.02 \text{ m}$

Take 3m x 3m





Assume load dispersion at an angle 45°

$\therefore$ D = 1.25m

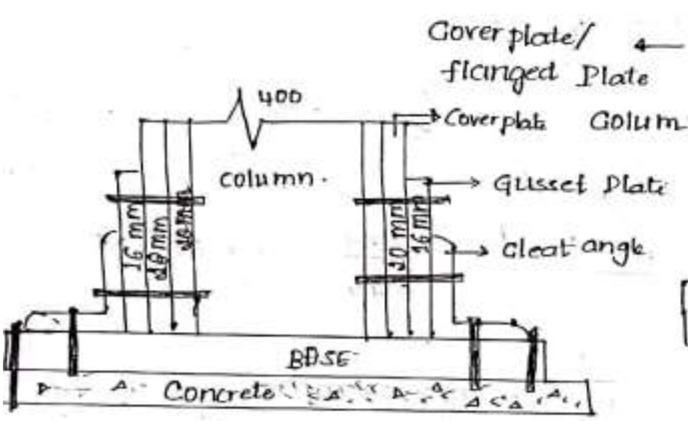
$\therefore$ $\boxed{\text{Concrete Base} \rightarrow 3\text{m} \times 3\text{m} \times 1.25\text{m}}$

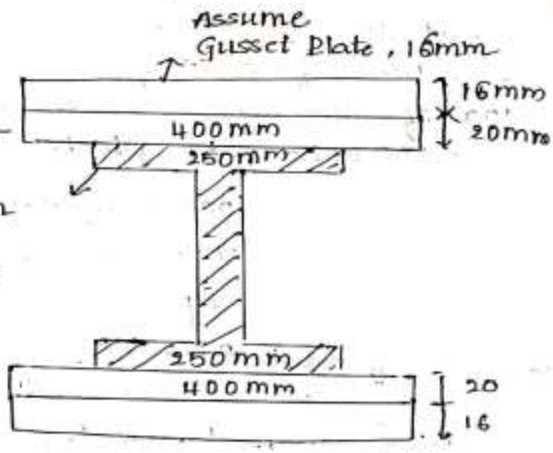
2. Design a gusseted base on a concrete pedestal for a column ISHB 400 @ 759 N/m with two flanged plates 400 x 20mm carrying a factored load of 4000 KN. The column is to be supported on concreted pedestal build with M20 concrete. Take SBC of soil as 225 KN/m^2

Soln: For ISHB 400 @ 759 N/m

$h = 400\,mm$; $t_f = 12.7$;

$b_f = 250\,mm$ $t_w = 9.1\,mm$





Size of cover plate / flange plate = 400 x 10 mm

Factored load = 4000kN

SBC of soil = 225 kN/m^2

$f_{ck} = 20\,N/mm^2$

a) Design of Gusseted plate (Area & Thickness)

$$Area = \frac{Load}{0.45\,f_{ck}} = \frac{4000 \times 10^3}{0.45 \times 20} = 444.44 \times 10^3\,mm^2$$

	Components	Length
1.	Column ISHB 400	400 mm
2.	Cover Plate (400 x 20)	2x20 = 40 mm
3	Assume Gusset Plate 16 mm Thick	2 x 16 = 32mm
4	Assume Cleat Angle 150 x 150 x10mm	2 x 150 = 300 mm

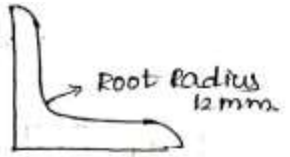
Total L = 772 mm

$\therefore$ Take $l = 800\,mm$

$\therefore l \times b = 444.4 \times 10^3$

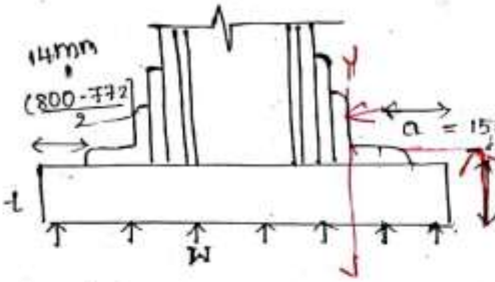
$800 \times b = 444.4 \times 10^3$

$b = 555.5\,mm$ say $560\,mm$

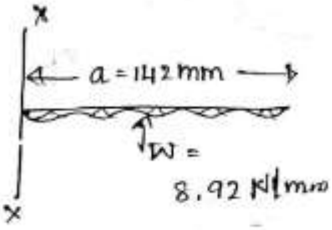


$\therefore$ Provide Gusset Base $l \times b = 800 \times 560\,mm$

b] Thickness of Gusseted Base.



$a = 150 + 14 - 10 - 12 = 142\,mm$.



Upward Pressure, $w = \frac{Load}{L \times B} = \frac{1000 \times 10^3}{800 \times 560} = 8.92\,N/mm^2$

Taking Considering 1mm strip, $\therefore b = 1\,mm$

$$M_{xx} = 8.92 \times 1 \times 142 \times \frac{142}{2} = 89.93 \times 10^3\,N\text{-}mm$$

Using $M = f_b \times Z$

$$= \left[\frac{f_y}{\gamma_{m0}}\right] \times \left[\frac{bd^2}{6}\right]$$

Here, $b = 1\,mm$
$d = (t + 10)$

$$89.93 \times 10^3 = \frac{250}{1.10} \times \frac{1 \times (t+10)^2}{6} \Rightarrow t = 38.72\,mm$$

Say, $\boxed{t = 40\,mm}$

c] Bolted Connection.

Using M-22mm property class 8.8 HSFG bolts

$\therefore$ Shear strength of HSFG bolts (bolt value)

$F_o = A_{nb} \times f_o$

$f_o = 0.78 \times \frac{\pi d^2}{4} \times 0.7 \times f_{ub}$

$= 172.2 \times 10^3$

$$V_{dsf} = \frac{1}{\gamma_{mf}}\left[\mu_f \times K_h \times n_e \times f_o\right]$$

$$= \frac{1}{1.25}\left[0.55 \times 2 \times 1 \times 172.2\right]$$

Bolt value, $V_{dsf} = 151.5$ kN

No of Bolt = $\frac{\text{Force}}{\text{Bolt Value}}$

Assume column bases are machine or grinded

$\therefore$ Load on flange $= \frac{4000}{2} = 2000$ kN

$\therefore$ load on each flange $= \frac{2000}{2} = 1000$ kN

No of Bolts $= \frac{1000 \times 10^3}{151.5 \times 10^3} = 6.6$

say 8 no of bolts.

d) Design of concrete base

Working load $= \frac{4000}{1.5} = 2666.67$ kN

Self Wt of column @ 10% of the working load.

$$\begin{array}{r} 2666.67 \\ 266.67 \\ \hline \end{array}$$

Total load $= 2933.34$

$\therefore$ Area of C.B $= \frac{\text{Total load}}{\text{SBC of soil}} = \frac{2933.34}{225} = 13.03\ m^2$

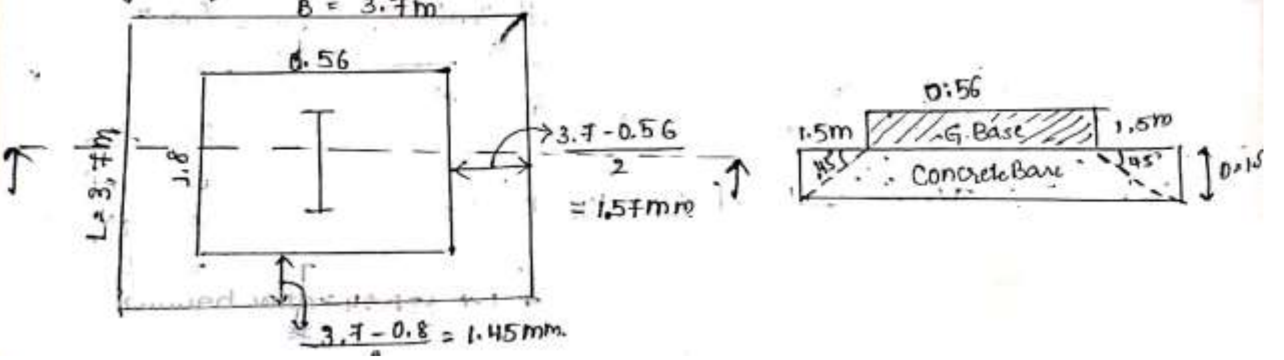
$\therefore$ Area $= L \times B = 13.03\ m^2$

Providing square base, side of C.B $= \sqrt{13.03}$

$= 3.62$ say

$= 3.7$ m

Depth of Concrete Base



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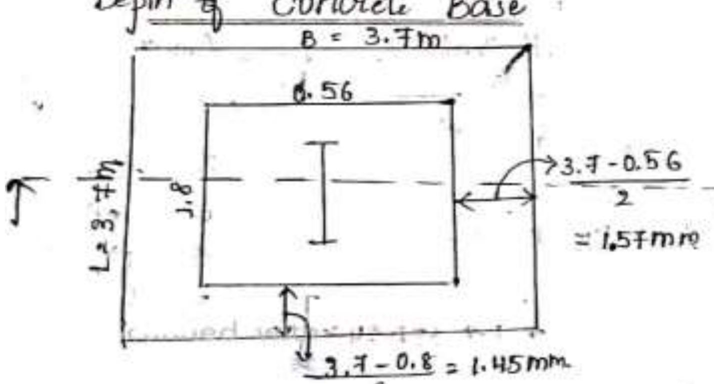
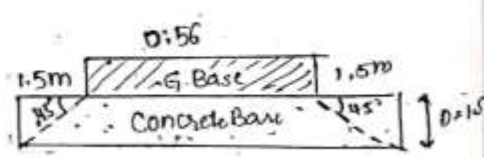
$\therefore$ Area $= L \times B = 13.03\ m^2$

Provinding square base, side of C.B $= \sqrt{13.03}$
$= 3.62$ say
$= 3.7$ m.

Depth of Concrete Base





DESIGN OF STEEL STRUCTURAL ELEMENTS (18CV61)

MODULE 5
DESIGN OF BEAMS

MODULE 05

DESIGN OF BEAMS

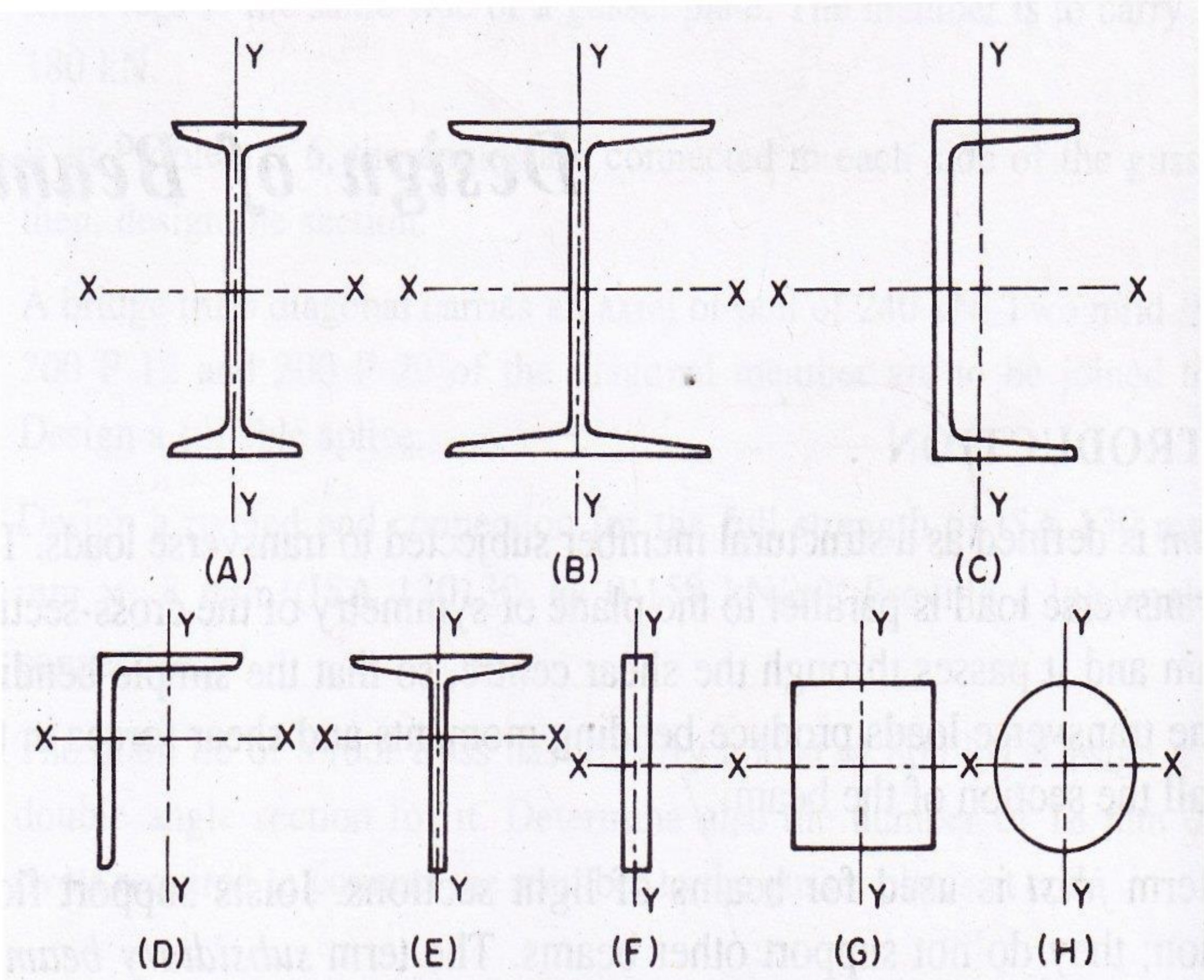
Introduction:

- ✓ Beams are structural members subjected to transverse loads in the plane of bending causing Bending moment and shear forces.
- ✓ These are horizontal structural elements that withstand vertical loads, shear forces, and bending moments.
- ✓ Beams transfer loads that imposed along their length to their endpoints such as walls, columns etc.
- ✓ The beams are designed for maximum BM and checked for maximum SF, local effects such as vertical buckling and crippling of webs and deflection.
- ✓ The compression flange of the beams can be laterally supported (restrained) or laterally unsupported (unrestrained) depending upon whether lateral supports (restraints) are provided are not.
- ✓ Beams can be fabricated to form different types of c/s for the specific requirements of spans and loadings.

Types of beam cross sections used in Steel Structures :

1. Rolled Sections /Universal Beam

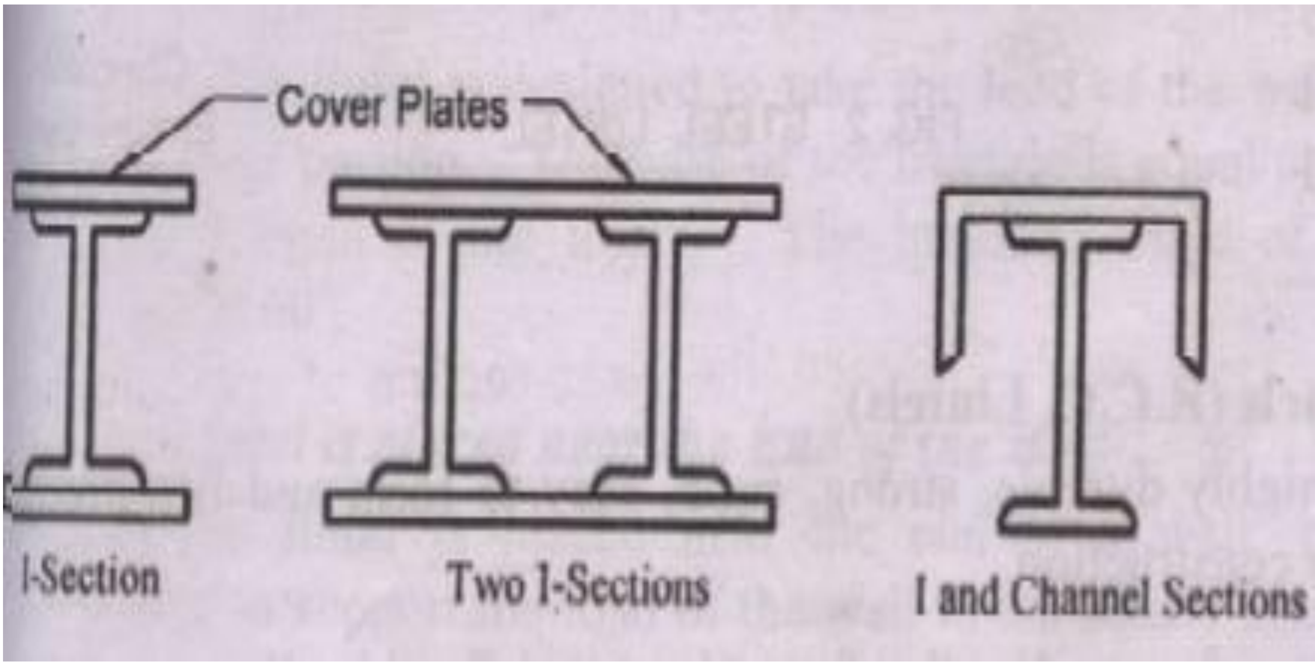
In this type material is concentrated in the flanges and are very efficient in resisting uni axial bending. Types of rolled sections used for beams are as follows

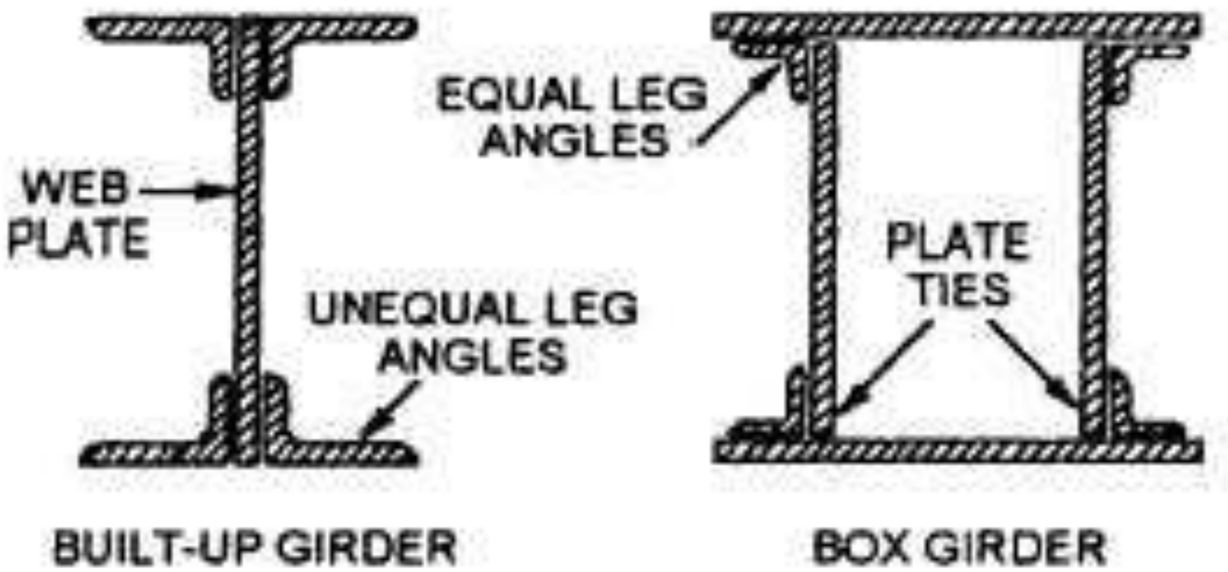


ROLLED STEEL SECTIONS USED AS BEAMS

2. Built-up Sections or Compound Beams :

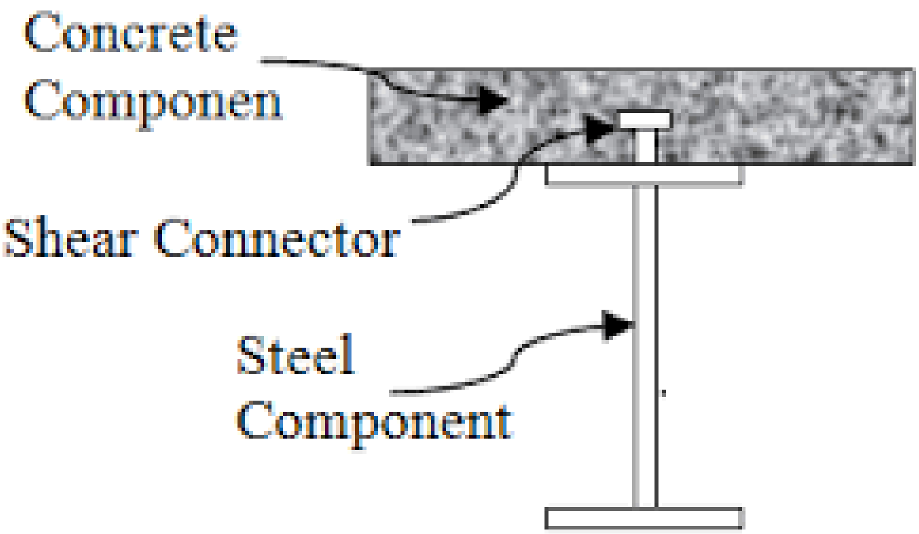
It consists of rolled sections strengthen by flange plates. This beams can resist bending in vertical as well as horizontal direction.

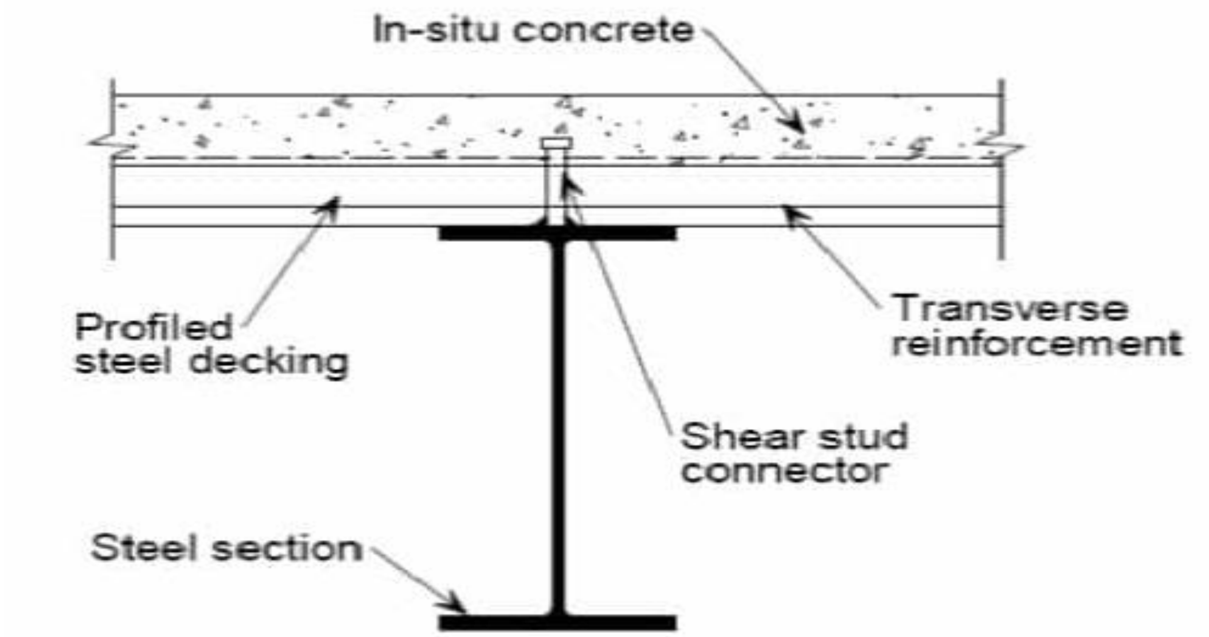




3. Composite Sections:

Composite beam consists of rolled section with roof slab which gives continues lateral support. The concrete floor over the beam provides the necessary lateral support to the compression flange to prevent lateral bucking.





Typical cross-section through a composite beam

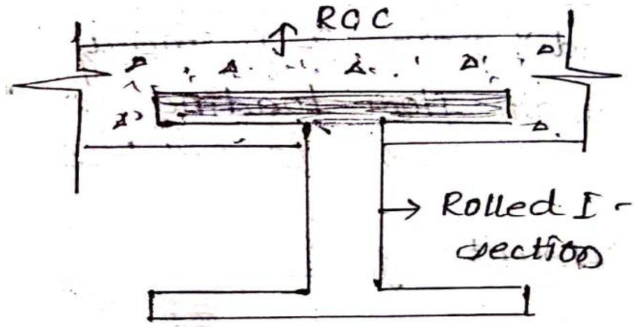
Section Classification:

There are four classes of section namely Plastic, Compact, Semi-Compact and Slender sections as per IS-800 : 2007 (Page 17). For design of beams, only Plastic and Compact sections are used.

Lateral Stability of Beams:

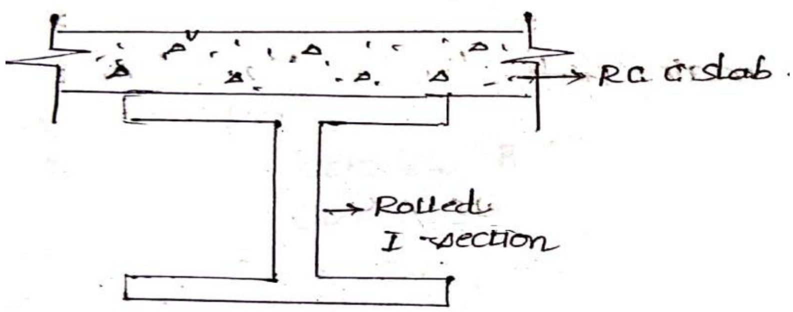
- A beam which does not laterally move nor rotate is known as Laterally-Supported Beam.
- Lateral buckling beams is the out of Plane bending due to compressive force in the flange and is controlled by providing sufficient lateral (support) restraint to the compressive flange.
- A laterally supported beam is one where the compression flange is supported and prevented from buckling in the horizontal plane due to the compressive forces in the top flange.
- This support could be in the form of a continuously welded chequered plate floor, or an RCC slab with shear lugs welded to the top flange of the beam or laterally supported by cross beams or bracings in the horizontal plane.

Difference between Laterally supported and Unsupported beams:



Laterally supported Beam (restrained)

In laterally supported beams, compression flanges are embedded in concrete and the beam is restrained (supported) against the rotation. Lateral deflection of compression flange does not occur.



Laterally Unsupported Beam (unrestrained)

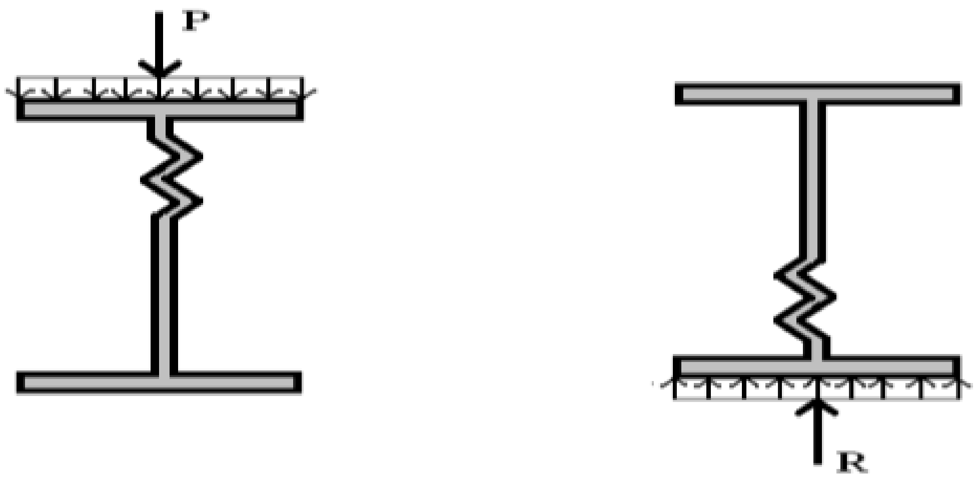
In laterally unsupported beam compression flanges are not embedded in concrete. Beam is free for rotation. Lateral deflection of compression flange occurs.

Factors affecting Lateral Stability of Beams:

The following factors affects lateral stability of beams

a. Cross sectional shape of the beam
b. Support conditions of the beam
c. Effective length of the beam
d. level of application of transverse loads.

Web Crippling (or Crimpling)



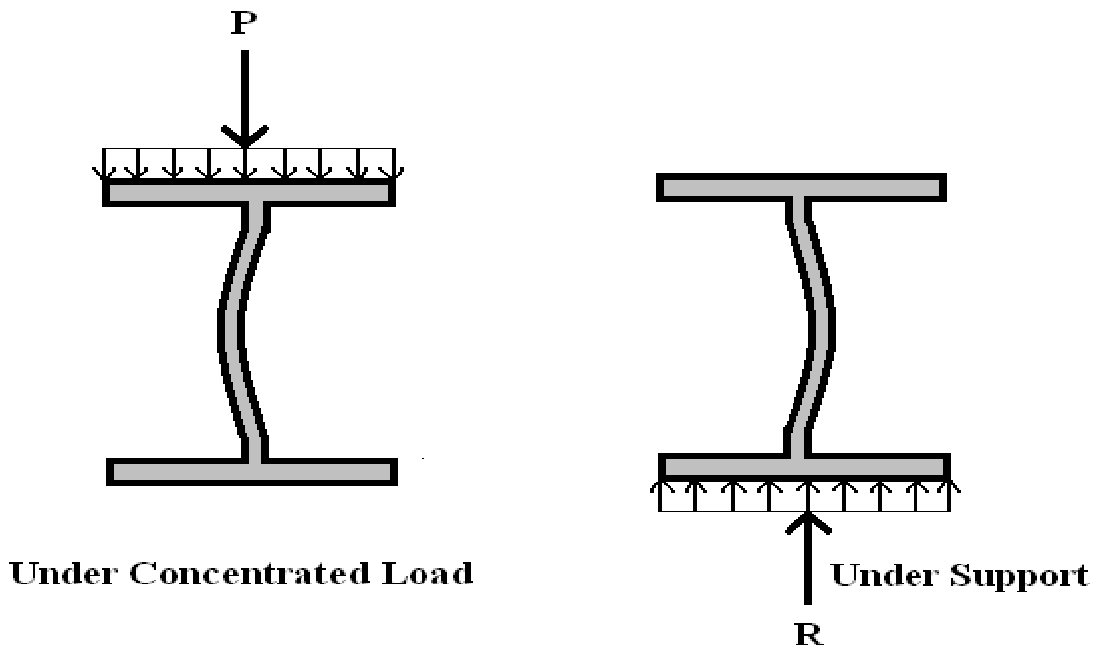
Under Concentrated Load **Under Support**

Web Crippling of Beams

- Web crippling causes local crushing failure of web due to large bearing stresses under reactions at supports or concentrated loads.
- This occurs due to stress concentration because of the bottle neck condition at the junction between flanges and web.

- ✓ It is due to the large localized bearing stress caused by the transfer of compression from relatively wide flange to narrow and thin web.
- ✓ Web crippling is the crushing failure of the metal at the junction of flange and web.
- ✓ Web crippling causes local buckling of web at the junction of web and flange.

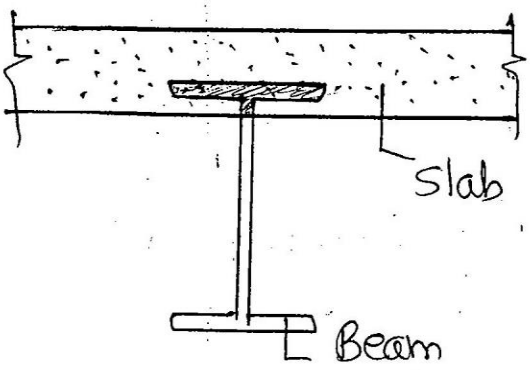
Web Buckling

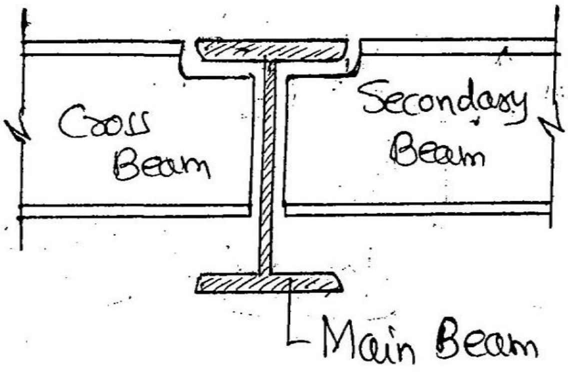


WEB BUCKLING OF BEAMS

- The web of the beam is thin and can buckle under reactions and concentrated loads with the web behaving like a short column fixed at the flanges.
- The unsupported length between the fillet lines for I sections and the vertical distance between the flanges or flange angles in built up sections can buckle due to reactions or concentrated loads. This is called web buckling.

DESIGN OF LATERALLY (RESTRAINED) SUPPORTED BEAMS



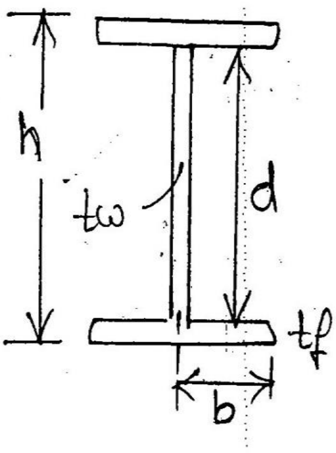


(a) Classification of Section : (Page 17 & 18)

	Plastic	Compact	Semi-Compact	Slender
(b/t_f)	9.4ε	10.5ε	15.7ε	$>15.7\varepsilon$
(d/t_w)	84ε	105ε	126ε	$>126\varepsilon$

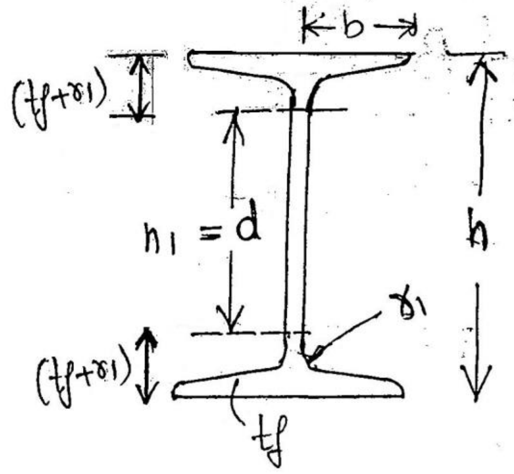
$$\varepsilon = \left(\frac{250}{f_y}\right)^{1/2} \quad \text{Usually } f_y = 250\,N/mm^2$$

$$\therefore \boxed{\varepsilon = 1}$$



$$\boxed{d = (h - 2t_f)}$$

or $$\boxed{d = (h - 2t_f - 2r_1)}$$



$\therefore d = h_1 = h - 2(t_f + r_1)$

or $d = (h - 2t_f)$

(b) "Moment of Resistance" of Beam

(i) If $\boxed{V_u \leq 0.6 V_d}$

Design Bending Strength $\Big\} = \boxed{M_d = \dfrac{\beta_b \cdot Z_p \cdot f_y}{\gamma_{mo}}}$ Page 53

$< 1.2\, Z_e \cdot f_y / \gamma_{mo}$ (Simply)

$< 1.5\, Z_e \cdot f_y / \gamma_{mo}$ (Cantilevers).

$\beta_b = 1$ (Plastic & compact)

$Z_p =$ Plastic modulus.

$\gamma_{mo} = 1.10$ (Table 5 Page 30)

(ii) If $\boxed{V_u > 0.6 V_d}$ Page 70

$\boxed{M_{dv} = M_d - \beta(M_d - M_{fd})} \leq 1.2\, Z_e \cdot f_y / \gamma_{mo}$

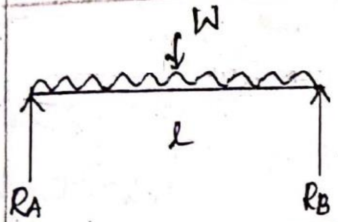
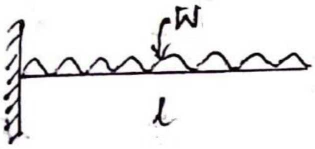
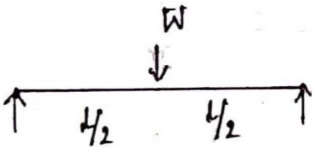
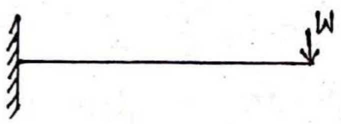
c) Check for deflection

$$\text{Permissible deflection} = \boxed{\frac{\text{Span}}{250}}$$

Permissible Deflection > Actual Deflection

NOTE:

The bending moment, shear force and actual deflection for various beams is as follows

		Bending Moment	Shear Force	Deflection
01		$M_u = \frac{w_u L^2}{8}$	$V_u = \frac{W_u L}{2}$	$\delta = \frac{5}{384} \frac{WL^4}{E_s I}$
02		$M_u = \frac{W_u l^2}{2}$	$V_u = W_u * L$	$\delta = \frac{1}{48} \cdot \frac{WL^4}{E_s I}$
03		$M_u = \frac{W_u L}{4}$	$V_u = \frac{W_u}{2}$	$\delta = \frac{1}{48} \cdot \frac{WL^3}{E_s I}$
04		$M_u = W_u L$	$V_u = W_u$	$\delta = \frac{1}{3} \frac{WL^3}{E_s I}$

$E_s = 2 * 10^5\ N/mm^2$

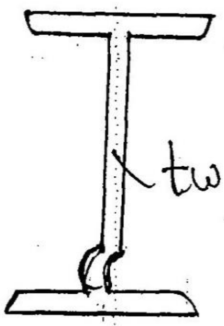
d) Check for "Shear" : (Page 59)

Design shear Strength $\Big\} = \boxed{V_d = 0.6\left[\dfrac{f_y}{\sqrt{3}\cdot\gamma_{mo}} \times A_v\right]} > V_u$

A_v = Shear area. = $h \times t_\omega$

(e) Check for "Web Crippling" : Page 67

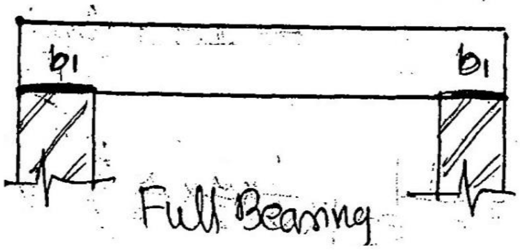
(Local failure)

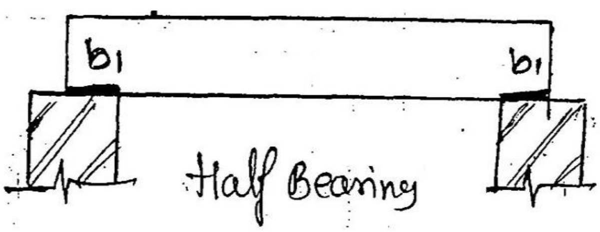


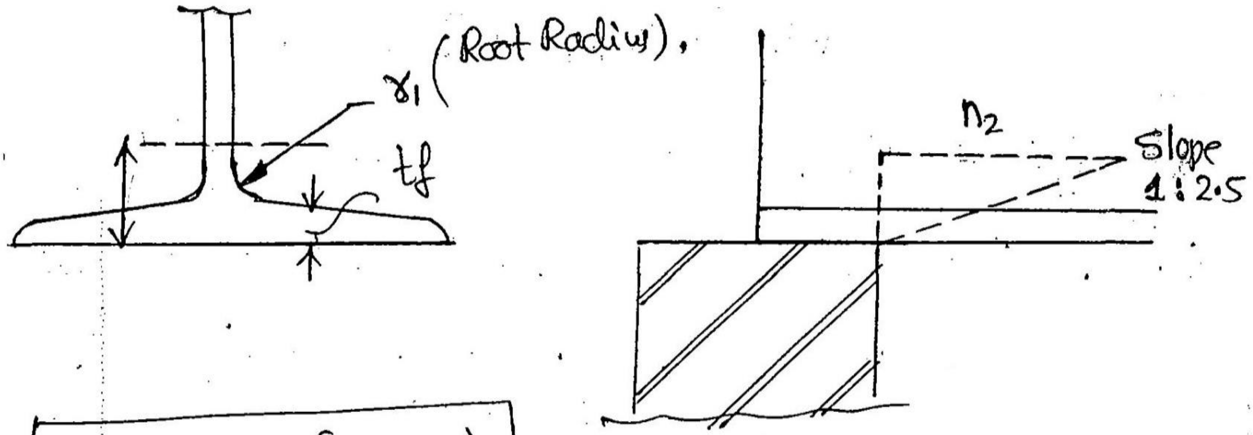
Bearing Strength

$$\boxed{F_\omega = (b_1 + n_2)\cdot t_\omega \cdot \frac{f_{y\omega}}{\gamma_{mo}}} > \text{Reaction.}$$

$f_{y\omega} = f_y = 250$ N/mm2

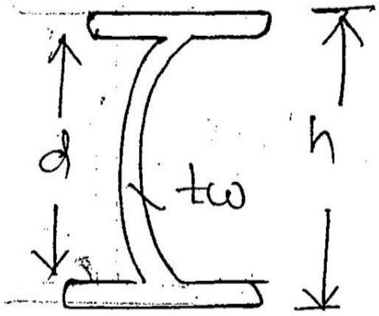






$\therefore \boxed{n_2 = 2.5(t_f + r_1)}$

(f) Check for "Web Buckling" (Page-67)



Buckling strength

$$\boxed{F_{wb} = (b_1 + n_1) \cdot t_w \cdot f_c} \rightarrow$$

★ To be Remember

$$n_1 = \frac{h}{2}$$

"f_c" → "Critical stress" from Table 9(c)

Page (42) (Design compressive stress = f_{cd})

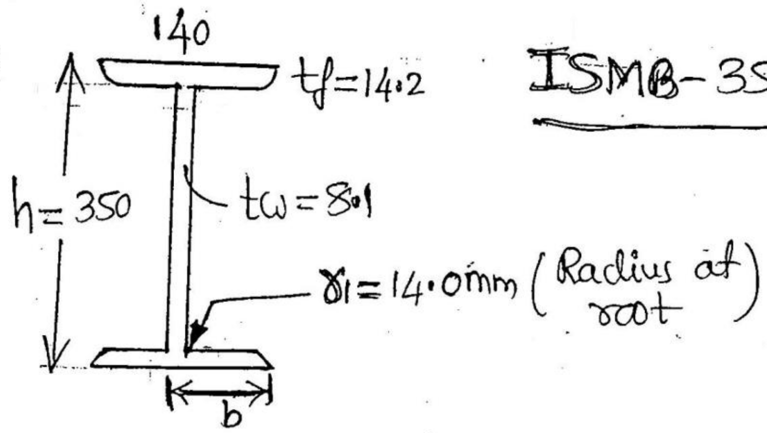
Slenderness Ratio = $\boxed{\lambda = 2.5\,\frac{d}{t_w}}$ ★ To be Remember

$$d = (h - 2t_f)$$

PROBLEM

1. Simply supported beam ISMB 350 at 52.4 Kg/m is used over a span of 5m. The beam carries an UDL live load of 20 KN/m and DL of 15 KN/m. The beam is laterally supported throughout. Check the safety of the Beam.

Sol^n



ISMB-350 @ 52.4 kg/m

(0.524 KN/m)

(a) Load calculation:

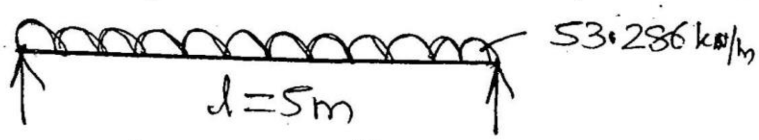
Self wt. of the beam = 0.524 KN/m

Dead load on beam = 15 KN/m

Live load on beam = 20 KN/m

$\overline{}$ 35.524 KN/m

∴ Ultimate load = 1.5 X 35.524 = $\boxed{53.286}$ KN/m



$M_u = \frac{wl^2}{8} = \boxed{166.5\ KN\text{-}m}$ $\quad V_u = \frac{wl}{2} = \boxed{133.21\ KN}$

(b) Check for "Shear"

Design shear Strength $\} = \boxed{V_d = 0.6\left[\frac{f_y}{\sqrt{3}\cdot\gamma_{mo}} \times A_v\right]} > V_u.$ Page 59

For Rolled section $A_v = h \times t_w = 350 \times 8.1$
$= 2835$

$\gamma_{mo} = 1.10$

$$\therefore V_d = 0.6\left[\frac{250}{\sqrt{3} \times 1.10} \times 2835\right] = \boxed{223.1\,kN}$$

$> V_u = 133.21$ kN (safe).

(c) Check for "Moment of Resistance" :-

$\boxed{V_u \leq 0.6\,V_d}$

Design Bending Strength $\} = \boxed{M_d = \frac{\beta_b \cdot Z_p \cdot f_y}{\gamma_{mo}}}$ Page 53

$$\left(\frac{b}{t_f}\right) = \frac{(140/2)}{14.2} = 4.92 < 9.4$$

$$\left(\frac{d}{t_w}\right) = \left(\frac{h-2t_f}{t_w}\right) = \left(\frac{350-2\times14.2}{8.1}\right) = 39.7 < 84$$

Page 18

$\therefore$ The Section is "Plastic" $\therefore \boxed{\beta = 1}$

From Table (46) Page – (138)

For ISMB – 350 → $Z_{P_z} = 851.11\ cm^3$

$= 851.11 \times 10^3\ mm^3$

$$\therefore M_d = \frac{\beta \cdot Z_p \cdot f_y}{\gamma_{mo}} = \frac{(1.0)(851.11 \times 10^3)\,250}{1.10}$$

$M_d = 193.43 \times 10^6$ N-mm $> M_u = 166.5$ kN-m (Safe)

(d) Check for "deflection" :-

$$\frac{\text{Span}}{250} = \text{Permissible deflection} = \frac{5000}{250} = \boxed{20mm}$$

Actual deflection:

$$\boxed{\delta = \frac{5}{384} \frac{wl^4}{E_s I_x}}$$

$E_s = 2 \times 10^5\ N/mm^2$

$I_{xx} = 13630.3 \times 10^4\ mm^4$
$= I_{zz}$

$$\delta = \frac{5(35.524^{*})(5000)^4}{384(2 \times 10^5)(13630.3 \times 10^4)} = 10.60mm < 20mm \text{ (Safe).}$$

* → The load should in Working (Not Ultimate)

(e) Check for "Web Crippling"

Bearing strength = $\boxed{F_w = (b_1+n_2)\,t_w \dfrac{f_{yw}}{\gamma_{mo}}} > V_u$

Assume bearing width $b_1 = 200mm$

$n_2 = 2.5(t_f + r_1) = 2.5(14.2+14)$

$= 70.5mm$

$\therefore F_w = (200+70.5)\times 8.1\,\dfrac{250}{1.10} = 497.96\,kN$

$> V_u$ (Safe)

(f) Check for "Web Buckling":

$\boxed{F_{wb} = (b_1+n_1)\cdot t_w \cdot f_c}$

Slenderness Ratio $\lambda = 2.5\dfrac{d}{t_w} = 2.5\dfrac{(h-2t_f)}{t_w}$

$\lambda = 2.5\dfrac{(350-2\times 14.2)}{8.1} = \boxed{99.25}$

From Table 9(c) $= 107\,N/mm^2 = f_{cd}$

$n_1 = \dfrac{h}{2} = \dfrac{350}{2} = 175mm$

$F_{wb} = (200+175)\times 8.1\times 107 = 325.01\,kN > V_u$ (Safe)

$\therefore$ ISMB-350 @ 52.4 kg/m $\rightarrow$ is Safe

Problems on Design of BEAM:

1. Design a beam of effective span of 6m subjected to an UDL 10 KN/m along with concentrated load of 100 KN at its centre. The beam is Laterally supported. The thickness of wall is 230mm.

Sol^n: **(a) Load calculation:**

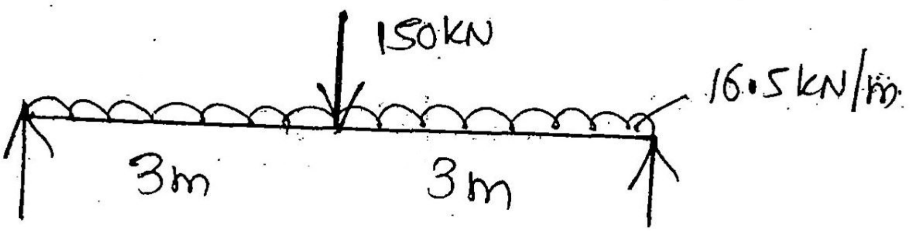
UDL on beam = 10 kN/m

Assume self wt. of beam = 1 kN/m

= 11 kN/m

∴ Ultimate UDL = 1.5 × 11 = 16.5 kN/m

& Ultimate Point load = 1.5 × 100 = 150 kN



$$\therefore V_u = \frac{16.5 \times 6}{2} + \frac{150}{2} = \boxed{124.5} \text{ kN}$$

$$M_u = \frac{16.5 \times 6^2}{8} + \frac{150 \times 6}{4} = \boxed{299.2 \text{ kN-m}}$$

$\therefore$ Plastic Modulus Required $\Big\} = \boxed{Z_p = \dfrac{M_d \cdot \gamma_{mo}}{\beta_b \times f_y}}$ mm^3 Page (53)

$$Z_p = \frac{299.2 \times 10^6 \times 1.10}{1 \times 250} = 1316.5 \times 10^3 \, mm^3$$

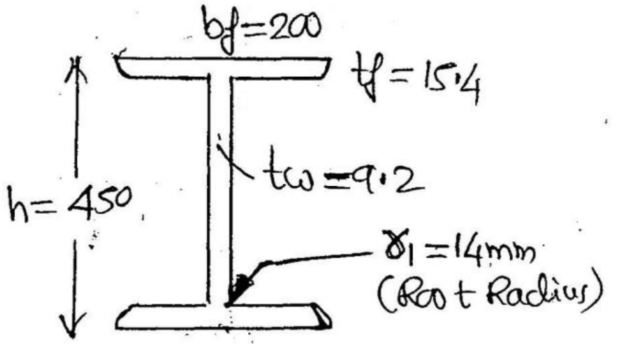
$$= 1316.5 \, cm^3$$

Increase the above value by 20% approximately

$$= 1.20 \times 1316.5 = 1580 \, cm^3$$

From IS-800 Page (138) Try

ISWB-450 @ 79.4 kg/m



$Z_p = 1760.59 \, cm^3$

$Z_e = 1558.1 \, cm^3 = Z_{xx}$

$I_{xx} = 35057.6 \, cm^4$

(b) Check for deflection

$$\text{Permissible} = \frac{\text{Span}}{250} = \frac{6000}{250} = \boxed{24mm}$$

Assume $E_s = 2 \times 10^5 \, N/mm^2$

$$\delta = \frac{5}{384}\frac{wl^4}{E_s \cdot I_x} + \frac{Wl^3}{48 E_s I_x}$$

$$= \frac{1}{(2\times10^5 \times 35057.6\times10^4)}\left[\frac{5(11^{*})(6000)^4}{384} + \frac{(100\times10^3)^{*}(6000)^3}{48}\right]$$

$$= \underline{9.06mm} < 24mm \text{ (safe)}$$

$*$ → The load for deflection check Should be in <u>Working</u> (Not Ultimate)

(C) Check for "Shear"

$$\left.\begin{array}{l}\text{Design shear}\\ \text{strength}\end{array}\right\} = \boxed{V_d = 0.6\left[\frac{f_y}{\sqrt{3}\times\gamma_{mo}} \times A_v\right]} > V_u$$

Page (59)

A_v → Shear area $= h \times t_w$

$= 450 \times 9.2 = 4140\ mm^2$

$$V_d = 0.6\left[\frac{250}{(\sqrt{3}\times1.10)} \times 4140\right] = 325.94\ kN \checkmark$$

$> V_u$ (safe)

(d) Check for M.R. :-

Section classification $\longrightarrow$ Table (2) Page (18)

$$\left(\frac{b}{t_f}\right) = \frac{(200/2)}{15.4} = 6.49 < 9.4$$

$$\left(\frac{d}{t_w}\right) = \frac{(h-2t_f)}{t_w} = \frac{(450-2\times 15.4)}{9.2} = 45.56 < 84$$

Hence the section is "Plastic" $\therefore \beta = 1$

$\therefore$ Design Bending Strength $\Big\} = \boxed{M_d = \frac{\beta_b \cdot Z_p \cdot f_y}{\gamma_{mo}}} > M_u$

$$M_d = \frac{(1)(1700.59\times 10^3)(250)}{1.10} = 400.13 \text{ kN-m} > M_u = 299.2$$

(Safe)

$$\& \quad \frac{1.2 Z_e \cdot f_y}{\gamma_{mo}} = \frac{1.2\times 1558.1\times 10^3\times 250}{1.10}$$

$$= 424.93 \text{ kN-m}$$

$$\therefore M_d < \frac{1.2 Z_e \cdot f_y}{\gamma_{mo}} \quad \text{(Safe)}$$

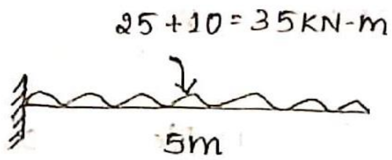
Check for Web crippling and Web Bulking can also be made as we done in the pervious problem.

2. Design a cantilever beam which is casted monolithic into concrete wall carrying a dead load of 25 KN/m and live load of 10 KN/m. Span of the beam is 5m.

Sol^n: Dead load = 25 KN/m

Live load = 10 KN/m

Span, L = 5m = 5000 mm.



a) Calculation of load.

Dead load = 25 KN/m

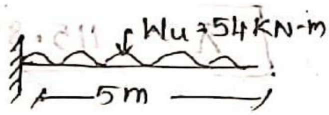
Assume self weight = 1 KN/m

Live load = 10 KN/m

Total UDL = W = 36 KN-m

∴ Ultimate UDL = W_u = 1.5 × 36 = $\boxed{54 \text{ KN-m} = W_u}$

b) Calculation of M_u & V_u



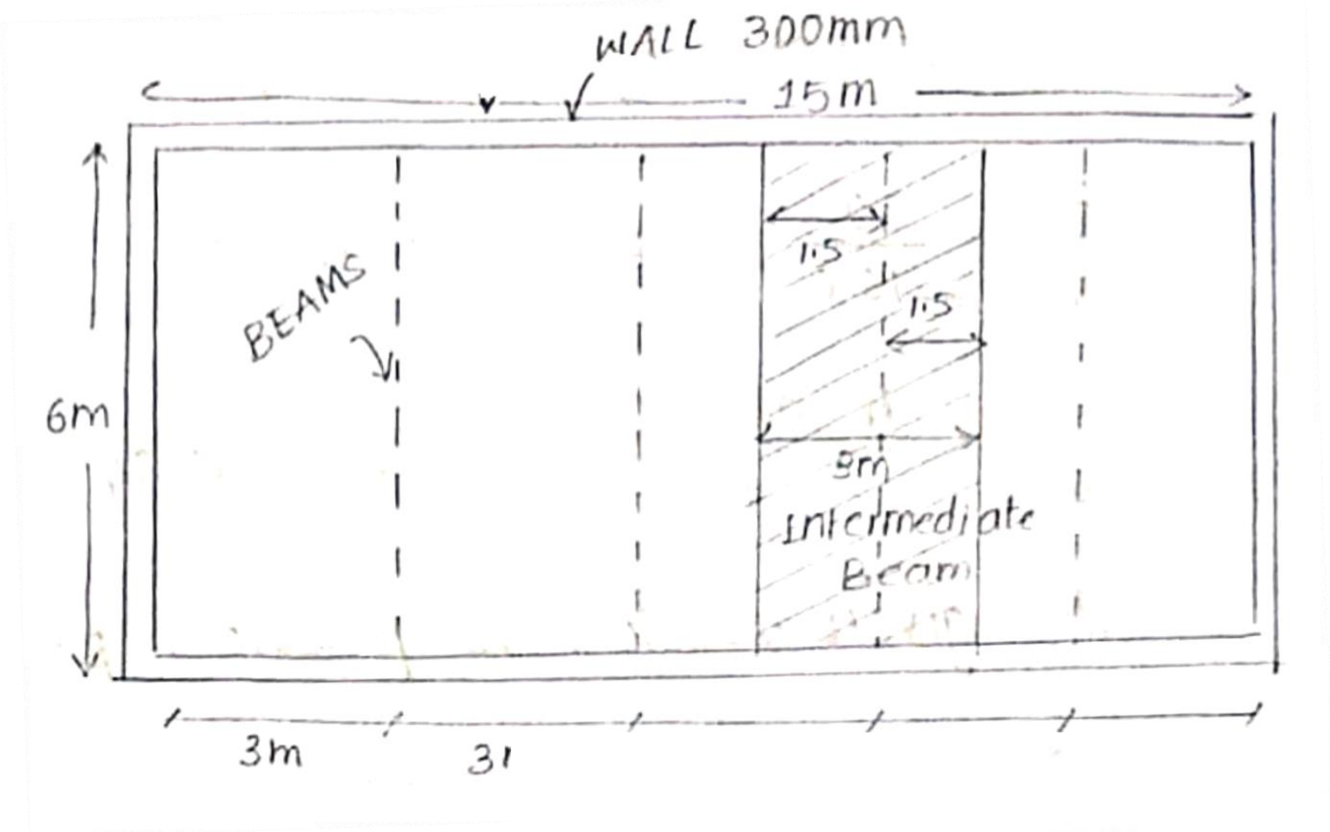
$$M_u = \frac{W_u L^2}{2} = \frac{54 \times (5)^2}{2} = \boxed{675 \text{ KN-m} = M_u}$$

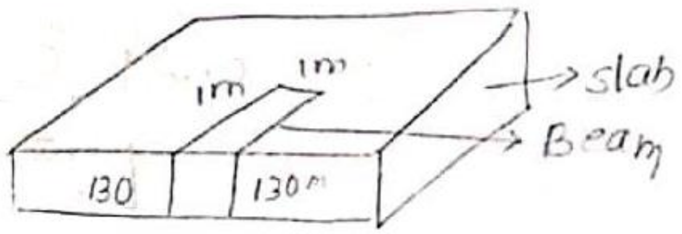
$$V_u = W_u * L = 54 * 5 = \boxed{V_u = 270 \text{ KN}}$$

c. Selection of trial Section based on Plastic Modulus Zp
d. Section Classification
e. Check for Shear
f. Check for Moment
g. Check for Deflection.

3. A hall measuring 6 m x 15 m consists of beams spaced at 3m center to center. RCC slab of 130 mm cast over the beams. The finishing load is 1.5 KN/m^2 and the imposed load on the beam is 5 KN/m^2 . The beam is supported on 300mm wall.

   Design an intermediate beam and check the design for deflection, web crippling and web buckling.





Sol^n: <u>Load Calculation</u>

[Considering 1m strip]

D.L of slab on Beam } $= (0.13 \times 1 \times 25) * 3$

$= 9.7\,KN/m$

Live Load $= 5KN/m^2 * 3 = 15\,KN/m$

Floor Finish $= 1.5\,KN/m^2 * 3 = 4.5\,KN/m$

Assume Self Wt $= 1\,KN/m = 1\,KN/m$

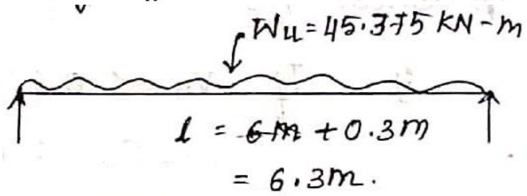
Total load, W $= 30.2\,KN/m$

$\therefore$ Ultimate UDL, $W_u = 1.5 * 30.25$

$$\boxed{W_u = 45.375\ KN\text{-}m}$$

b] <u>Calculation Maximum shear force & Bending Moment</u>

$$\therefore M_u = \frac{W_u l^2}{8} = \frac{45.375 * (6.3)^2}{8}$$



$$\boxed{M_u = 225.11\ KN\text{-}m}$$

$$V_u = \frac{W_u l}{2} = \frac{45.375 * (6.3)}{2} \Rightarrow \boxed{V_u = 142.95\ KN}$$

c] <u>Selection of section [Pg. 53]</u>

Using $M_d = \dfrac{\beta_b Z_p f_y}{\gamma_{m0}}$ &

Equating Design Moment = Max BM

$M_d = M_u$

$$225.11 * 10^6 = \frac{1 * Z_p \times 250}{1.10}$$

$$\boxed{Z_p = 990.4\ KN\text{-}m}$$

Increase approximately by 20%.

$$Z_p = 1.2 \times 990.4\ KN\text{-}m$$

$$\boxed{Z_p = 1188.48\ cm^3}$$

Try ISMB 400 @ 61.5 kg/m $\therefore$ $Z_p = 1176.18 \times 10^3\ mm^3$

$Z_e = 1020.0 \times 10^3\ mm^3$

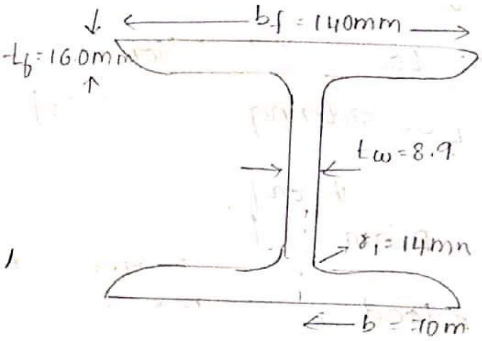
$I_{xx} = 20458.4 \times 10^4\ mm^4$

d) section classification.

$h = 400\,mm$; $b_f = 140\,mm$; $t_f = 16.0\,mm$; $t_w = 8.9$; $r_1 = 14\,mm$

$$\frac{b}{t_f} = \frac{70}{16.0} = 4.375 < 9.5\,\epsilon$$

$$\frac{d}{t_w} = \frac{h - 2(t_f + r_1)}{8.9} = 38.2 < 84\,\epsilon$$



$\therefore$ The given section is plastic $\therefore \beta = 1$

e) Check for shear [Pg.59

$$V_d = 0.6\left[\frac{f_y}{\sqrt{3}\,\gamma_{mo}} \times A_v\right] > V_u$$

$A_v = h \times t_w$
$= 400 \times 8.9$
$= 3560\,mm^2$

$$= 0.6\left[\frac{250}{\sqrt{3} \times 1.10} \times 3560\right]$$

$$\boxed{V_d = 280.27\,KN > V_u}$$

f) Check for Moment

i) If $V_u \le 0.6\,V_d$ [Low shear]

$142.95 \le 0.6 \times 280.27$

$142.95 \le 168.162$

$$\therefore \quad M_d = \frac{\beta_b f_y Z_p}{\gamma_{mo}} > M_u \le \frac{1.2\,Z_e f_y}{\gamma_{mo}}$$

$$= \frac{1 \times 250 \times 1176.18 \times 10^3}{1.10} > M_u \le \frac{1.2 \times 1020 \times 10^3 \times 250}{1.10}$$

$\boxed{M_d = 267.31\,KN\text{-}m >}$ $225.11\,KN\text{-}m < 278.18\,KN\text{-}m$

$\therefore$ safe.

g) Check for deflection

$$\delta_{permissible} = \frac{span}{250} = \frac{6300}{250} \Rightarrow \delta_{per} = 25.2\,mm$$

$$\delta_{actual} = \frac{5}{385} \times \frac{WL^4}{E_s I_{xx}} = \frac{5}{385} \times \frac{30.25 \times (6.3)^4}{2 \times 10^5 \times 20458.4 \times 10^4}$$

$$\boxed{\delta_{actual} = 15.16\ mm}$$

$\delta_{actual} < \delta_{per}$

Hence safe.

h) Check for Web crippling [Pg. 67]

$$F_w = (b_1 + n_2)\, t_w \times \frac{f_{yw}}{\gamma_{m0}} > V_u$$

$b_1 = \frac{300mm}{2} = 150mm$

$n_2 = 2.5(t_f + r_1)$

$= 75mm$

$$= (150+75) \times 8.9 \times \frac{250}{1.10} > V_u$$

$$\boxed{F_w = 455.11\ KN > V_u}$$

Hence safe

i) Check for Web buckling

$$F_{wb} = (b_1 + n_1)\, t_w f_c > V_u$$

$n_1 = \frac{h}{2} = \frac{400}{2} = 200\,mm$

$b_1 = 150\,mm$

$\lambda = 2.5\frac{d}{t_w} = \frac{2.5 \times (400-2(16+r_1$

8.9

$\lambda = 95.50$

∴ From IS code table 9C [Pg. 42]

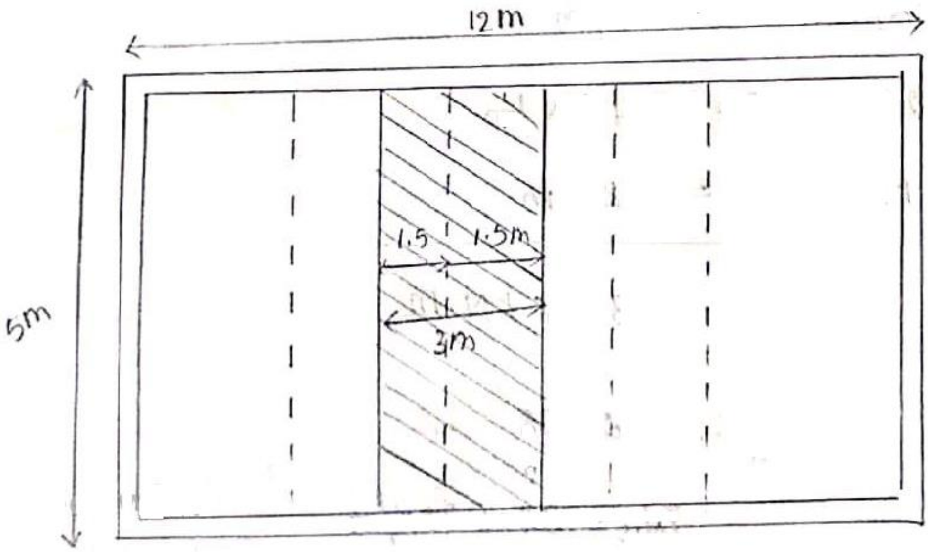
90	121
100	107
95.5	113.3

$$F_{wb} = (150+200) \times 8.9 \times 113.3$$

$$\boxed{F_{wb} = 352.92\ KN\text{-}m\ V_n}$$

Hence safe ∴ Adopt ISMB 400 @ 61.5 kg/m

4. A roof of hall measuring 5 x 12 m consists of 120 mm thick RCC slab on steel I-section spaced at 3m centre to centre. Take live load of 3.5 KN/m^2 and finishing load 1.5 KN/m^2. Bearing of wall 400 mm. The beam is laterally restrained
Design one of the interior beam supporting the roof, Check for shear, moment capacity and Deflection.



a) Load Calculation

Dead load	=	$(0.12 * 1 * 25) * 3$		= 9 KN/m
Live load	=	3.5 KN/m^2	* 3	= 10.5 KN/m
Floor finish	=	1.5 KN/m^2	* 3	= 4.5 KN/m
Self wt	=	1 KN/m^2	* 1	= 1 KN/m
		Total load		25 KN/m

Ultimate load = $1.5 * 25 \Rightarrow W_u = 37.5$ KN/m

Length = 5m + 0.4m = 5.4m.

b) Calculation of Maximum shear & Bending Moment

c) Selection of trial section based on Zp.

d) Section classification

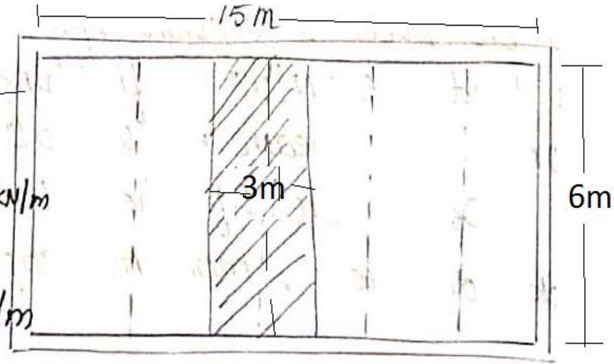
e) Check for shear

f) Check for moment

g) Check for deflection.

5. A hall of clear dimensions 15 m x 6m is to be covered by RCC slab flooring of 120 mm thick resisting over RS joist spaced at an interval of 3m c/c. Floor finishing is 20mm thick is to be provided over the RCC slab. The live load on RCC slab is 4 KN/m^2. The joists are resting over 300 mm thick wall.

Design an Engineering beam using code specification. The unit Weight of RCC and Concrete is 24 KN/m and apply all



a) Load Calculation.

DL of slab on beam $= 0.12 * 24 * 3 = 8.64$ KN/m

LL $= 4$ KN/m$^2 * 3 = 12$ KN/m

Floor finish $= 0.02 * 24 * 3 = 1.44$ KN/m

Assume self Wt $= 1$ KN/m $= 1$ KN/m

$W = 23.08$ KN·m

$W_u = 1.5 * 23.08$

$W_u = 34.62$ KN/m

b) Calculation of Max S.F & B.M

c) Selection of trial section based on Z_p

d) Section classification

e) Check for shear

f) Check for moment

g) Check for deflection

h) Check for Web crippling

i) Web Buckling.